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What Has Happened to Water Works Construction?

By Harry E. Jordan

THE American Water Works Association at midyear in 1943 joined with the New England Water Works Association, the Federation of Sewage Works Associations and the Water and Sewage Works Manufacturers Association in forming the Committee on Water and Sewage Works Development. Abel Wolman acted as chairman of the committee, which was made up of two representatives from each of the constituent associations.

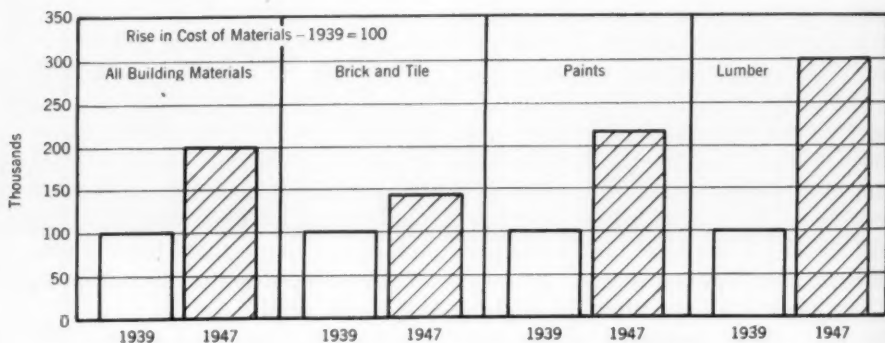
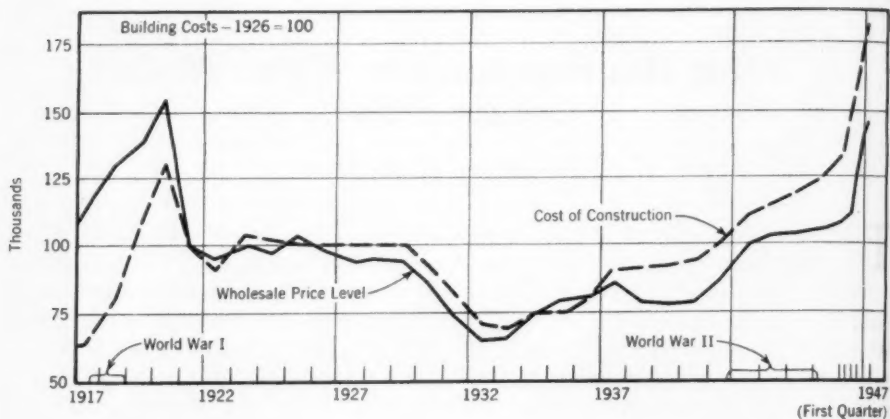
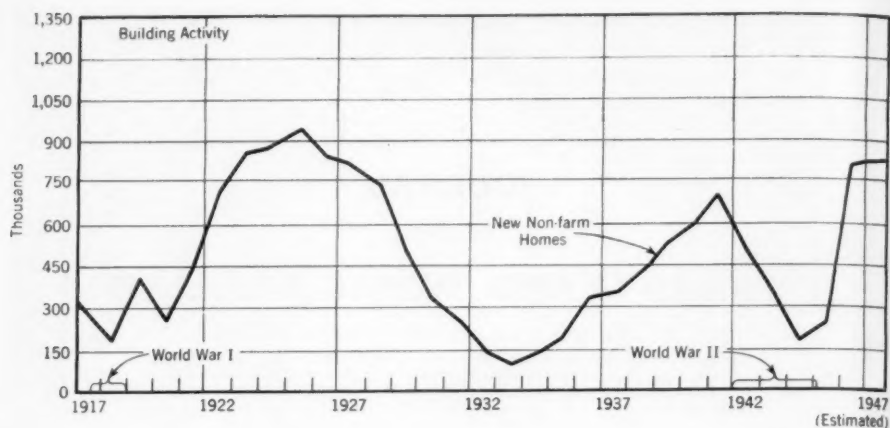
The idea back of the organization of the committee ran parallel to the idea which prompted the A.S.C.E. to organize a postwar planning committee, and which prompted the various industrial leaders to organize the Committee on Economic Development.

The idea was that, after the end of the war, large numbers of persons would be unemployed unless advance planning was done to provide both pub-

lic and private construction. Since basic engineering as well as industrial construction not directly related to war production had been laid aside during the war, all recognized that the backlog of needed construction would be very great. What all persons felt was needed was "plans ready for contract" at the end of the war.

The promotional work which was done by the Water and Sewage Works Committee was fruitful. At midyear 1945—after VE day—water and sewage works plans in progress totaled an estimated cost of \$1,500,000,000 out of a total of \$8,000,000,000 for all public works. This water and sewage total was more than three times its normal proportion of the nation's public works total.

After VJ day in August 1945, the expected mass decline in employment did not develop. The alarmist predic-



Courtesy Pictograph Corp. and N.Y. Times

FIG. 1. Housing Construction and Building Costs

tion of 8 to 10 million unemployed was not fulfilled. Industry reconverted rapidly. Employment of labor shifted from war to peacetime production. Competition for construction materials became fiercer. Prices began to rise. So, at the end of 1945, the Committee on Water and Sewage Works Development made a decision which was not received with favor in some quarters. It decided to cease its promotion—upon the theory that intensified planning of public works would add to the inflationary trend. The committee did not disband. It stands by at the present time, awaiting evidence that it can again serve the country and the water and sewage works field by promotion of construction.

The committee has not advised against the development of plans for works for which the need is current and imperative. It has not advised against long-range planning to the contract document stage—that is, readiness to initiate construction when conditions are propitious. It may be noted that consulting engineers at midyear 1947 are busy in development of plans for water and sewage works.

In 1946 Howson predicted that postwar construction costs would not go below $1\frac{1}{2}$ to $1\frac{1}{2}$ times prewar costs. *Engineering News-Record's* Construction Cost Number shows that April 1947 construction costs are four times 1913 costs and $1\frac{1}{2}$ times 1944 costs. The trends taken by construction costs and the amount of building activity over a period of years are shown in Fig. 1. At what level stabilization will develop in the construction industry is the nation's question.

A few weeks ago a prominent manufacturer in the water works field offered his opinion that construction in the water and sewage works field was not progressing because: (1) increased costs of operation had consumed the reserves set aside for postwar construction or (2) municipalities were raiding water works reserve funds and converting them to general uses.

These opinions appeared to merit discussion and it was decided to inquire what appraisal the various State Sanitary Engineers made of the suggestions of the manufacturers.

The inquiry was initiated on April 25, 1947. The replies received up to May 30 are briefed in the Appendix to this paper (p. 610)—in the order of their writing. This series of comments is intensely interesting and of great value. It shows that rising costs of construction are, in the opinion of the state engineers, the dominant factor causing deferment of postwar construction. The replies give little evidence of new "raiding" of water works funds, but they do show that the diversion of water works funds into general municipal uses is carried on widely, and is too often done without real consideration of the long-term needs of the water works system.

The tenor of the replies makes evident the propriety of the Association's again addressing itself to the promotion of orderly and planned contribution to city general funds, measured either in tax terms or in terms of a limiting percentage of the net revenue of the city-owned water works system. This will undoubtedly be an A.W.W.A. project in the immediate future.

APPENDIX

Replies of State Sanitary Engineers

Louisiana

The two primary causes of deferment of many postwar projects are price increases and material shortages, and of course these are related.

We have no information at present of any raiding of water department funds by city administrations in Louisiana.

John H. O'Neill

West Virginia

It is my opinion that price increases are largely responsible for the deferment of many postwar projects. These increases are of course brought about to some extent by the material shortage.

It has been the practice in some of the communities to use surplus water funds for other administrative purposes.

J. B. Harrington

Alabama

A number of water and sewage works projects are being planned at the present time; many of them are in the design stage. Unless conditions make it absolutely necessary that projects be undertaken, particularly in the larger municipalities, the officials, often upon the advice of their engineering consultants, are withholding construction due to high costs and also to the uncertainty of the delivery of materials. A number of relatively large projects are under construction, however, and several of the smaller municipalities are now undertaking the installation of complete new water works systems, including sources of supply. The larger projects that are under construction

were considered urgent projects, as the present facilities are inadequate to meet the demands.

I feel sure that the money made available by the federal government for plans and surveys has stimulated the planning of many municipal projects, including water and sewage works. There is a possibility that many of these projects that have been undertaken with federal aid will become a reality when construction costs are cheaper.

In Alabama, for the most part, water works management is under the direct jurisdiction of the municipal governing bodies, such as the commission or city council. In the majority of these plants, a very small per cent, if any, of the revenue is set aside for expansion of water works facilities or for maintenance. This situation is unfortunate, but does exist. Many of our municipalities, on the other hand, have placed the operation of the water works under a water works board, and, in these plants, water works operation is much more efficient. The trend of water works management seems to be in this direction, brought about possibly by the need of finding ways and means of financing needed improvements.

Practically all construction that has been undertaken in Alabama in the past several years has been financed by the sale of revenue bonds. This type of bond has found a ready market. It is our understanding that the bonded indebtedness of the municipality does not limit the financing of projects by this method.

Arthur N. Beck

Georgia

There is no more evidence in Georgia of "raiding" of water works and sewage funds at the present time than in the past. It is always a potential hazard and frequently requires vigilance and competence in practical politics on the part of the water and sewage officials to prevent such inroads. The reverse procedure may be equally true. It is a case of the pot and the skillet within the municipal family of service agencies, all deserving a share of the total income of taxes, charges and fees. Many water and sewage officials do not perform *cum laude* in this form of competition. My opinion is that the percentage of total municipal income obtained by water works has not changed materially in recent years.

Operating and constructing facilities within the framework of unchanged—or moderately increased—income from water and sewage charges, or other sources, is the rub. Considerable upward revision in service charges to meet the present cost situation seems inevitable. The picture painted during the war of large reserves being accumulated for future construction was largely erroneous. Such reserves, where they exist, will hardly take care of deferred maintenance at present costs.

Price increases and material shortages are universal. In addition to these general conditions, the manifestation of them when the construction project comes to the contract-letting stage is even more pointed. In every contract drawn up in Georgia within the last year, the consulting engineers have estimated on the upward curve of costs, then added an amount for the uncertainties of everything up to a point just below where the municipality throws up its hands in total discouragement. With bated breath one then approaches

the few sealed envelopes containing "bids" and finds that, during the advertisement period, the slope of the cost curve has increased from 45 to 90 degrees.

The reason for excessive costs given by the few contractors at all interested in bidding are often very real and quite understandable. In general, it is not only the high cost and shortage of materials but the *uncertainty* about what the final price will be, since the escalator is being worked overtime. Add to this the situation, in this region at least, that many contractors apparently ended the war period of construction in such a favorable financial condition that they have no incentive to take risks now. The income tax "take" further removes incentive for a reasonable venture. One could go on and on in pointing out unusual factors.

On the constructive side of the picture, it is becoming increasingly clear that water works and sewage men must adjust their thinking and action to what is unusual to them, and learn to cope with the situation as it is and not as they would like to have it. There are ways open to the ingenious and aggressive individual to keep his foot in the door, and construction has already begun to move where there is a strong guiding mind. Although certainly not moving at the pace everyone expected, a number of large and small projects are under construction in Georgia. They simply could not wait longer. Necessity and ingenuity combined to make for progress.

W. H. Weir

New York

In the past there was less tendency for municipalities to divert funds from water supply accounts to general accounts than at the present time, but we

are under the impression that the increased expenditures by municipalities incidental to rising prices is now leading to a wider use of water funds for general municipal purposes.

This situation was brought to our attention recently when we learned that the city of Auburn had in its water funds \$165,000 worth of government bonds and \$50,000 in cash, which had been accumulated for water supply improvements. Cash to the amount of \$50,000, however, has been transferred from the water fund to the general municipal fund, and it is understood that in the future all excess earnings of the water department will be made part of the general fund.

Upon the receipt of this information, we conferred with the office of the State Comptroller and we learned that Sec. 18 of Art. III of the State Constitution prohibits the legislature from passing any law preventing the use for general municipal purposes of any profits or surplus resulting from the operation of a public utility. We know of no way, therefore, in which this diversion of the funds can be stopped except by the slow process of education.

C. A. Holmquist

New Hampshire

The only water works construction projects scheduled for this year are those that are more or less of an emergency nature. In other words, these jobs are the ones that cannot be deferred any longer. There is a large reservoir of work piling up, not only for water works but also sewerage systems and sewage treatment plants. For the most part, these projects have been deferred until the prices are considerably reduced. The material shortage is annoying, but is not nearly so much a factor as are the high prices.

In this state, legislation was passed during the war to allow cities and towns to build up a sinking fund. A part of this law specifies that this reserve must be set aside for specific purposes. At this present session of the legislature, the sinking fund provision has been made a permanent law, and, therefore, a number of cities and towns are holding their money, as provided by the law, to be spent for specific things, such as extension of mains, building of new sources of supply, construction of filtration systems and so on.

Leonard W. Trager

New Jersey

The primary reasons for the deferment of the many postwar projects in New Jersey are price increases and the fact that the financial status of many of the municipalities is such that, in order to construct the projects, they must exceed their legal borrowing capacity. It is true that during the past 10 years the gross municipal debt has been decreased (according to the reports of the New Jersey State Local Government Dept. this decrease amounts to 35 per cent), but usually this reduction was insufficient to permit the undertaking of new water works and sewerage projects. Many projects have been deferred because of delay in delivery of materials. It is believed, however, that if prices were within reason the time limit would be of secondary importance.

We cannot express an opinion about the general tendencies to divert the water department financial reserves by city budget authorities for other purposes. It is known, however, that several water department funds were raided by city administrations.

J. Lynn Mahaffey

Mississippi

There is no doubt in my mind that the present material and construction costs in the water works field are definitely retarding the letting of contracts in this state. There is a sizeable backlog of construction work already planned, but few contracts are being let, not so much due to the inability of the municipality to finance the works, but because of the excessive costs and their hopes for lower costs in the future. Except where the need is very urgent, construction work is being delayed, and on some contracts all the bids that were received have been rejected.

There has been no evidence to my knowledge of municipalities raiding the water department funds for general administration purposes. It is also my opinion that most of the municipalities in Mississippi are in good financial condition, and there definitely has been a lowering of the bonded indebtedness during the past half dozen years. I feel confident that, if the cost picture for water works construction would improve slightly in the near future, a considerable number of our municipalities would proceed with their proposed works.

H. A. Kroeze

Tennessee

Work has not yet started on many projects in Tennessee. The major causes of the delay have been price increases, material shortages and the present over-all cost of construction, compared to estimates made when the projects were planned. Also, there are many communities who are holding back in the hope that there will be federal aid for the construction of projects. As a matter of fact, many of the projects were planned with the ex-

pectation that there would be a federal subsidy for construction.

We do not have a record of the bonded indebtedness of the municipalities or knowledge of their method of financing operation except insofar as the matter may be discussed in an informal way at the time of our routine investigations. We have no specific knowledge, however, of accumulated surpluses being used for municipal operations.

R. P. Farrell

North Dakota

It is my opinion that there is a more or less general tendency to raid water department and other utility funds by many of the cities in North Dakota. This raiding has for years been limited by law to 10 per cent of the gross income of the utility, but was raised to 20 per cent during the last session of the legislature. There is considerable question in my mind that the 10 per cent limitation was strictly honored, and, no doubt, there are also ways and means of getting around the 20 per cent figure.

Many postwar projects, of course, are being deferred because of material shortages and increased prices. Several small municipalities have submitted plans and specifications for complete water and sewage systems, but find it impossible under present conditions to start construction. Those projects that are being constructed are financed by revenue bonds, general obligation bonds and special assessments. Rarely has any appreciable reserve been built up in anticipation of this work.

As a general statement, I believe that most North Dakota municipalities could, by some means or other, finance the needed improvements to their municipal utilities if they so desired,

but it may not be considered good business to do so now.

Jerome H. Svore

Idaho

The two principal causes for deferment of many postwar projects are price increases and material shortages.

It has been our observation that the *Engineering News-Record* construction cost index for April 1947 of 396.09 is somewhat low for construction costs in Idaho. At present it is true that funds which have been set aside for postwar construction are inadequate to do work then planned, and upward revision of funding or downward revision of plans must be effected.

The delay in delivery of water works material has seriously discouraged executives from 1947 construction.

City indebtedness in Idaho has been decreased notably since 1940. Many cities are now debt-free, and some have acquired funds which may be used for new construction or deferred maintenance.

It has been our observation that raiding of water department funds by city administrations in Idaho is commonly practiced.

H. C. Clare

New Mexico

Procurement of materials and high costs are the major difficulties. In two water supply projects it has been necessary to build one earth dam and enlarge another, in order to impound a greater amount of raw water, and to defer filter plant construction and reconstruction a year until materials become more readily available and more bonds can be voted. There seems also to be a slight feeling—or perhaps a strong hope—that by 1948 construction prices may come down a bit and deliveries improve.

Many cities and towns have materially reduced their bonded indebtedness during the prosperous war years. Others have authorized water and sewer bond issues, but have not sold the bonds. A few have sold bonds and construction is under way. In New Mexico bond proceeds may not be diverted to other purposes than those for which they were appropriated. It is possible that some water works revenues may have been temporarily or possibly permanently diverted to meet increasing operating costs in other departments of municipal government, but this appears to be largely a passing tendency which will not adversely affect water and sewage works interests once construction can get under way again. Of course I may be mistaken about this, but I feel that our water works are getting so badly worn, so badly behind with much-needed extensions and improvements that, once materials become easier, there will be such an overwhelming demand for funds that any diverted water and sewage funds will be forthcoming.

Warren H. Book

Virginia

A number of communities in Virginia have completed plans for construction of water and sewage works. Bonds have been issued for many of the projects, but, because of the tremendous increase in prices, advertising for bids has been withheld.

Considerable maintenance work is being done, but it is being stretched over long periods because of delays in obtaining the material.

I have no knowledge of funds set aside for water and sewage works being diverted to finance current municipal operating expenses.

Richard Messer

Montana

We, out here, are confronted with the serious condition of price increases. Some engineers who made estimates three or four years ago now find themselves somewhat embarrassed, especially if bonds had already been issued on their estimates.

The delay in delivery of water works materials has also affected planning adversely. Some municipalities are going ahead anyway. Others have been discouraged from making major improvements by delays in delivery.

There has always been some tendency in Montana to use revenues obtained by the water departments for other city expenses. This practice assumed rather serious proportions some years ago, but has not been so serious in more recent times. I have talked to municipal engineers about this and find that there seems to be a difference of opinion. One engineer is rather strongly of the opinion that, as the water system is owned and operated by the municipality, it is not so serious if the funds which are found in excess are used for other municipal expenses.

Perhaps this is not serious if the water department still retains enough money for all debt services and good operating conditions, and has a comfortable reserve for emergencies and needed improvements. With the increased demand for public service on the part of our city dwellers, and with a restriction upon the levying powers of the municipality, there no doubt is a temptation to use reserves which may be in any other funds. There are some legal restrictions on this, however, and doubtless they serve as a brake upon the practice.

Although municipal councilmen and other officials are pessimistic about the costs of municipal government, Mon-

tana municipal officials on the whole are cheerful and fully aware of their obligations, and are willing to strain every effort to meet the demands and still keep budgets balanced.

The public water supply needs in Montana at this time are largely for expansion of distribution systems and for repairs required by the necessary neglect of the systems during the past few years. The largest expenditures in the immediate future will be for repairs, so far as present indications show.

H. B. Foote

Oklahoma

According to the information received from the water superintendents, the bonded indebtedness in many communities has been reduced since 1941, and many communities have voted new bond issues to cover the cost of proposed improvements. These improvements have not been made, mainly because of the increased cost of both materials and labor. An interesting sidelight on this point is the trend that some of the communities are following: to purchase the materials on the open market and then perform the work themselves. Several cities have purchased war surplus equipment for their construction projects, but, contrary to the original intent, are retaining this equipment for future use.

Many communities have experienced difficulty in raising sufficient funds to operate the numerous activities of city government. It is possible that funds from the water departments have been diverted to support other activities, but I do not think that it is a general policy throughout the state.

There are a number of projects which we should like to see under construction, but, after discussing them with city officials, it was decided to withhold

any action until there was a possibility of getting contracts within the limits of the funds available.

H. J. Darcey

Arkansas

There has been considerable postwar planning of water works in this state, but little construction. It is doubtful if many of the planned projects will be completed in the near future. The high costs and uncertainty of materials have caused a halt in construction. When bids are taken the prices quoted reflect the high labor cost and the uncertainty of delivery of materials. In numerous cases the costs to the city for improvements at this time are prohibitive.

In a few towns, the water works bears the cost of the sewage treatment plant and maintenance and operation for the system. There are no known cases of outright misappropriation of funds set up for water works construction and operation.

It is hoped, as deliveries of materials become more certain and prices drop, that many of the planned facilities can be constructed.

F. L. McDonald

Massachusetts

Nearly all municipal water works in Massachusetts are established under special state laws which contain various provisions for the financial organization of the water works. Under Ch. 396 of the Acts of 1928, all provisions of such special acts enacted prior to 1921 which authorized water departments to spend money for the water works without appropriation by the municipality were repealed.

According to the Div. of Municipal Finance of the Dept. of Corporations and Taxation, approximately one-half of the special acts provide for a re-

stricted use of water revenue in some what the same manner as the following

... The income of the water works shall be appropriated to defray all operating expenses, interest charges and payment on the principal as they accrue upon any bonds or notes issued for the purpose of a municipal water supply. If in any year there should be a net surplus remaining after providing for the aforesaid charges for that year, such surplus, or so much thereof as may be necessary to reimburse the town for moneys theretofore paid on account of its water department, shall be paid into the town treasury. If in any year there should be a net surplus remaining after providing for the aforesaid charges and for the payment of any such reimbursement in full, such surplus may be appropriated for such new construction as the water commissioners, or selectmen authorized to act as such, with the approval of the town, may determine upon; and in case a net surplus should remain after payment for such new construction the water rates shall be reduced proportionately. . . .

Otherwise, and this has been the trend in recent special acts establishing water works, all water receipts are used as "estimated receipts" toward a reduction of the tax rate. The water departments may use only amounts appropriated by the city or town, and the entire revenue goes into the general funds. A water works department in this position might be "raided" by being required to operate on appropriations voted while turning over its receipts to the town. There seems to be no evidence of any wholesale "raiding" of funds, but on occasion a very considerable portion of the revenue has been used by municipalities for other than water supply purposes. Often, however, the water department itself was at fault because of its failure to prepare a proper budget of expenditures. Many

in some water departments, more by habit than following for any other reason, submit a budget of about the same proportions as for the previous year. Such budgets seldom recognize the necessity to make replacements in the distribution system and to finance the construction of additional sources of supply, as water consumption demands require. If the departments had careful engineering investigations and advice in such matters, they would appreciate the necessity of budgeting in an entirely different manner, and there is every reason to believe that the municipality would back them up if the information were properly presented at the time the appropriations were requested.

There is considerable discussion at present on the question of whether water departments should make annual payments to the municipality "in lieu of taxes" on an agreed basis. Holyoke has such a provision in its act, and in addition pays rent to the city for quarters occupied in the city hall.

Arthur D. Weston

Texas

Although we are not intimately in contact with a large number of municipalities, we do not believe that there is any large-scale "raiding of water department funds by city administrations." It is our feeling that the apparent reluctance of municipalities to enter into large improvement programs at this time can be primarily attributed to two uncertainties: the uncertainty of delivery of necessary materials and equipment, and the uncertainty of the price of these materials when they are delivered. These two factors, coupled with the cost increase in labor, in our opinion, are the major factors affecting construction programs.

V. M. Ehlers

Florida

Our municipal situation is not good. Citizens are demanding increased services on the one hand and tax cuts on the other. In addition, an increasing number of revenue-producing means are being reserved for higher government levels.

In Florida, it is not only common but *general* practice to divert income from municipal water and other utilities—quite often to the extent that entirely inadequate financial reserves are maintained. Of course, there are exceptions, but they are unusual.

Water system construction and expansion are being delayed in Florida for two reasons: (1) uncertainty about the future and inadequate supplies of required materials, and (2) what, in our opinion, is an exceedingly regrettable attitude on the part of municipal officials to continue to hope for federal financial assistance and, hence, to delay desirable construction pending the possible proffer of such aid.

In addition, we must fight continuously (within the limited resources available to us) to develop an appreciation at the municipal level of the necessity of providing adequate funds in the water works financial structure to insure proper operation—excluding maintenance.

We believe that in Florida, water works financial reserves are depleted or non-existent, and only the most urgently needed water supply construction is under way or contemplated for 1947. Financing of needed work will obviously be based largely on revenue certificates.

David B. Lee

Oregon

Public water supplies in Oregon may be divided into three categories:

1. Municipal supplies operated under a water commission with authority separate from that of the city government

2. Municipal supplies in large cities

3. Municipal supplies in small cities.

The water departments in large cities and water works which are operated under separate water commissions have established reserve funds, and, to our knowledge, these funds are set aside for the maintenance and expansion of their individual water supply systems. No attempt has so far been made to utilize these reserves for any other purpose.

In our smaller cities, water revenues are generally placed in the general fund and, when extensions or major improvements are required, bonds are issued to finance the work. Water department revenues are then used to retire the principal and interest on this bonded indebtedness. During the war years many smaller communities were able to establish reserves which are now being used to extend the existing distribution systems to meet current demands.

Although it is true that price increases and material shortages have temporarily deferred some water works improvement projects, many of our water districts and municipalities are proceeding with construction work at the present time to increase their supplies and to provide water service in new areas. In order to meet the current increase in operating costs, some of our municipalities have increased their water rates correspondingly, and, so far as we have been able to determine, the financial status of Oregon's municipal water supplies is in excellent shape.

There may, of course, be isolated instances where water department revenues deposited to the general funds in a city may be used to defray other op-

erating costs. These, however, are the exception rather than the rule.

C. M. Everts, Jr.

South Dakota

Many previously planned postwar projects were deferred, originally because of shortage of materials. That shortage still exists in several items; however, in the meantime construction costs have risen to the point where available funds are not now adequate to finance the proposed construction. Many cities in this state are postponing contemplated construction at least until 1948, in the hope that there will be some decrease in the cost of construction, together with an increase in the contractors' labor efficiency which will permit them to submit lower bids.

Funds that were set aside or authorized by municipal elections for bond issues to finance various sanitary improvements are now generally not adequate to construct the plants as originally planned. It is not believed possible for most municipalities to raise additional funds through bond issues, but if construction is started, they will probably delete certain plant improvements in the hope that these can be financed through plant revenues at a later date.

Many South Dakota municipalities are finding it difficult to provide for sufficient operating funds, and some water revenues have been diverted to the general fund. This has always been the practice in some cities. In general the indebtedness of the municipalities was considerably decreased from 1939-40 to 1945. After 1945, however, several municipalities voted to bond themselves for various improvements, a move which may result in greater indebtedness now than was experienced during the early forties.

W. W. Towne

Organization and Financing in Indiana

By Otto K. Jensen

A paper presented on May 8, 1947, at the Indiana Section Meeting, Indianapolis, Ind., by Otto K. Jensen, Executive Secretary, Indianapolis Redevelopment Commission, Indianapolis, Ind.

FOR the past few months the author has been a member of a panel sponsored by the Indiana Economic Council which has conducted many discussions on municipal planning, trying to acquaint the citizens with the importance of planning on a community basis, so that first things may be done first and the plans adopted may be within the ability of the taxpayers and ratepayers to finance. It was also pointed out that most Indiana communities are blessed with much government—that is, with many overlapping agencies and departments that have debt-creating authority—and, if each agency proceeded with the projects it wanted, the financial burden would be too great for the citizens to bear. There is a real need for co-ordinating planning, so that what is wanted may be reconciled with what is needed and with the citizens' ability to pay. Every community should have its 5- or 7- or 10-year program of capital improvements.

Efficient Management

All planning, however, should be based on a sound and efficient operating policy. A water utility is "big business" in its community, and deserves good management.

Efficient management will not result from a policy of political employment without regard to ability to perform the job assigned. In many cities and towns

the entire personnel changes with each political change in the municipal administration; such a policy increases the operating expenses of the utilities and impairs the efficiency of operation. Merit employment is much to be preferred in the operation of utilities, with proper safeguards for the employer and retirement provisions for employees.

Another laxness in some communities is that of proper collection procedures and policy. A utility must have rules and regulations that are carried out without favor or interference. Too often little effort is made, for example, to keep collections current.

Additional abuses are those brought about by the city or town administration that asks the utility to furnish materials, supplies and often labor without reimbursement; or that passes an ordinance to transfer the cash balance of the utility—as a "surplus"—to the municipal general fund for civil operations.

Some utilities have a provision for hydrant charges in their rate schedules, yet no cash receipts are obtained for this purpose; some enter the item and periodically charge off the account as uncollectable. It is not being urged that a municipally owned utility should make no payment to the civil unit, but only that such payments should be measured by the payment in taxes which that utility would pay were it privately owned.

If a utility is to set up the funds required under the Indiana statutes for (1) operation, (2) depreciation and (3) debt service, it is necessary to have: efficient management with capable employees; a rigid policy for collecting bills, including hydrant charges; adherence to the principle that no purchases will be made for others without proper reimbursement, nor transfers made to the civil unit unless there is an actual surplus; and a policy that no surplus exists unless an adequate depreciation fund exists, to be used for improvements and extensions. Had such a policy of operation and allocation of cash receipts been in practice during the past several years, problems of funds for extensions and improvements at this time would be minimized. Utilities which have operated on such a basis are now in a preferred position to proceed with their improvements and, if necessary, with public borrowings.

Revenue Bond Issues

The question of public borrowings, these days, more and more often focuses upon the revenue bond. This type of bond has come into use in the United States during the past 50 years. Only a few were issued prior to 1915, and more general use has been seen only since 1933. Although the earlier revenue bond issues were for the establishment of water supply systems, later issues were for financing such enterprises as electric light and power systems, bridges, toll highways, transportation systems, hospitals, stadiums, swimming pools and sewage disposal systems. With proper enabling legislation, almost any public project may be financed by revenue bonds "if the project will produce things or provide services for which the public will pay and if sufficient revenue can be obtained from the

project by the sale of such things or services."

Many municipalities have determined to issue revenue bonds because the general obligation debt was so large that enough money could not be borrowed for the project without exceeding the constitutional debt limitation. Revenue bonds are not affected by the usual debt limitation.

A search of the Indiana statutes discloses the following water works revenue statutes:

1921. Chapter 96, Acts of 1921, supplemented by Ch. 56, Acts of 1925, amended by Ch. 190, Acts of 1927, which was again supplemented by Ch. 88, Acts of 1929, as amended by Ch. 287, Acts of 1947 (Sec. 48-5345 *et seq.* Burns 1933):

Authorizes any city, town or other municipal corporation, subject to approval of the Public Service Commission (except where bonds are for acquisition and earnings are adequate under Sec. 48-5348 Burns 1933), to issue revenue bonds to purchase and acquire, or to construct extensions, additions and improvements to water works to supply such city, town or municipal corporation and the inhabitants thereof with water for public and domestic use.

1929. Chapter 155, Acts of 1929, amended by Ch. 254, Acts of 1933 (Sec. 48-5328 *et seq.* Burns 1933):

Authorizes any city or town owning and operating unencumbered water works, supplying such city or town and the inhabitants thereof with water for public and domestic uses, to construct extensions and improvements to such water works and to finance the cost thereof by issuance of revenue bonds, subject to approval of Public Service Commission.

1933. Chapter 125, Acts of 1933, amended by Ch. 311, Acts of 1935 and Ch. 107, Acts of 1939 (Ch. 125, Acts of 1933 amended Ch. 77, Acts of 1929, which

supplements Cities and Towns Act of 1915) (Ch. 129, Acts of 1905) (Sec. 48-7119, 48-7101, 7103 Burns 1933):

Authorizes cities of the first class (Indianapolis) to issue utility revenue bonds payable solely and exclusively from the income and revenues of such utility, as defined, with which to provide funds to pay for the acquisition of any utility property to redeem or extinguish capital stock of any utility, etc., and to make necessary betterments, improvements, extensions or additions to such utility property.

1933. Chapter 190, Acts of 1933, amended by Ch. 293, Acts of 1935 (Ch. 190, Acts of 1933, amended Chapter 76, Acts of 1913, the "Shively-Spencer Utility Commission Act") (Sec. 54-105, *et seq.* Burns 1933):

Authorizes any municipality (city or town), without consent or control of any department, bureau or commission, other than the municipal council of the municipality, to own, lease, erect, establish, purchase, condemn, construct, acquire, hold and operate any utility, and to finance the cost thereof by the issuance of revenue bonds.

1933. Chapter 235, Acts of 1933 (Sec. 48-5301 *et seq.* Burns 1933):

Authorizes any city or town owning and operating a water works to create a department of water works and issue revenue bonds, subject to the approval of the Public Service Commission, to finance the construction of extensions, additions, betterments and improvements thereto.

1933. Chapter 259, Acts of 1933 (Sec. 48-5441 *et seq.* Burns 1933):

Authorizes any city of the fifth class owning and operating unencumbered water works supplying such city and the inhabitants thereof with water for public and domestic use, to issue revenue bonds, subject to the approval of the Public Service Commission, to finance the construction of extensions, additions and

improvements to such system. Compare *Long v. Stemm* (1937) 212 Ind. 204, for determination of bad title and reliance upon Ch. 235, Acts of 1933, as an alternative method of proceeding.

1937. Chapter 206, Acts of 1937 (Sec. 61-508 *et seq.* Burns 1933):

Authorizes refunding, refinancing and improving of any utility owned by any municipality.

1939. Chapter 107, Acts of 1939 (Sec. 48-7120 *et seq.* Burns 1933):

Authorizes cities of first class to purchase the whole or part of any utility and to issue revenue bonds under any relevant act.

It appears that there are ample provisions for the municipally owned utilities to avail themselves of the use of revenue bonds. In the author's opinion, they should be used with great discretion, and many bond issues in the past should not have taken place—some because the plant could not be constructed within the proceeds of the bond issue, others because preliminary surveys were not adequate to determine a sufficient source of supply, and still others because a study of customers' income was not made to indicate whether revenue would be sufficient to guarantee payment of operating expense and bond and interest payment, without prejudice to a proper depreciation fund.

Until 1943 another serious defect in revenue bond sale procedure was that the authorization, execution and sale of revenue bonds often occurred at a single meeting of the legislative body of the city or town. The law now requires advertisement and public sale of revenue as well as general obligation bonds.

Several questions need consideration before a unit of government issues its revenue bonds:

1. Can the community afford the project? A thorough study must be made of this point. Customers are usually the taxpayers of the community, and their income cannot be overburdened. There is no magic in revenue bonds which makes their payment painless.

2. Do the results of engineering surveys and estimates support the project? The importance of the engineering expert cannot be stressed too strongly. The best consultant is none too good, and may well prove the most economical in the long run. Although the opinion of the expert may be needed, however, little reliance should be placed upon opinions that are not based on adequate factual investigation.

3. Does the community support the project wholeheartedly? This aspect of an enterprise is overlooked, yet it is essential to the success of the project. Opportunity should be provided for thorough discussion, based upon a clear statement of the results of factual investigation.

4. Have all legal requirements been complied with? Like the engineering service, expert legal advice is essential. Too often a lack of legal knowledge of procedure has resulted in delays and increased expense.

5. Has a good prospectus been prepared? Many local governments are not doing a good job of publicizing bond issues. The bond buyer must have the facts about a city before he is willing to bid a rate commensurate with the risk. It is essential to acquaint prospective buyers with the city, the essentials of the project, an economic survey by competent authority and

population trends; and to supply them with testimonial letters from business leaders, copies of good annual reports, informative current financial statement, and data on the source of support of both payment of interest and the retirement of principal of the bonds, plus the fact that careful engineering estimates adequately support the anticipated revenue.

The aim should be to reach many prospective bidders, and not surround the sale with secrecy in order that a few bidders may be favored.

All these ends require careful financial planning so that the best interest of the taxpayer and ratepayer may be safeguarded. The past can bring forth countless examples of bad financial planning, as the often cited example of the most costly municipal bond on record—issued by a large city in 1870 for \$4,500, carrying 7 per cent interest and maturing in 2146.

Conclusion

Financial soundness and maximum efficiency require a well-managed plant and proper allocation of revenues to permit a pay-as-you-go policy wherever possible. When borrowings are necessary, however, expert services should be engaged to determine the economic soundness of the project, obtain community support and prepare an informative prospectus that will impress the bond buyers and secure the lowest financing costs obtainable for the community. When such precautions are taken, the water works executives will be able to plan carefully and wisely, construct well, and pay promptly.

Methods of Fixing Public Fire Protection Charges

By Fred R. Witherspoon

A paper presented on May 8, 1947, at the Indiana Section Meeting, Indianapolis, Ind., by Fred R. Witherspoon, Senior Water Works Engr., Public Service Commission of Indiana, Indianapolis.

THE measures to be employed in establishing charges for public fire protection are a matter that concerns all water utility operators, with their more or less fixed incomes, in the present period of constantly rising costs. The problem becomes peculiarly more complicated because, if the funds are to come from a tax levy, the utility must contract with public officials, who themselves are affected by similar economic conditions. An accumulation of long-deferred public improvements, brought about by war conditions, and shortages of material and labor, are making exceptionally heavy demands on budgets of public expenditures. All these trends, moreover, are taking place in the face of an unpredictable future.

The primary purpose of a water utility is to provide for the health, safety and welfare of the inhabitants of a community. This service is normally divided into two major classes: water for general consumption, and fire protection needs. The fire protection service, with which this discussion is concerned, is the service performed by a water utility in providing a reliable source of supply, pumping equipment, storage facilities, transmission and distribution mains, and hydrants and appurtenances adequate to supply fire-fighting apparatus with all the water required at any time, even under con-

ditions of maximum demand by general water consumers. Fire protection needs are considered on the basis of plant capacity surplus and readiness to serve over and above maximum general consumer requirements.

A fair charge for fire protection service would be an equitable return on a fair value for that portion of a water works property used in providing the service. It is important that a study be made to determine the total charges allocable to fire protection service, in order to produce this equitable return. The mechanics of applying the charge in the form of hydrant charges, charges per mile or inch-foot of mains or additional levies on consumer billings can then be worked out to produce the total revenue allowable from this source.

Arbitrary Charges

A review of the operations of about 300 water utility properties in Indiana makes it apparent that no large amount of study has as yet been given to the total amount of fire protection charges or how these charges should be applied. General practice in Indiana is to apply them through the annual fire hydrant charge. It is surprising how often this public fire hydrant charge is arbitrarily fixed by some sort of a capricious agreement between the utility and

municipal officials. Often, when tax reduction is mandatory, or money is needed for a more urgent purpose, city budget makers will employ the hydrant charge money to meet expenses elsewhere.

It may be of interest to cite a water rate case heard by the Indiana Public Service Commission a short while ago. The petitioner's witness, an executive of one of the largest water utilities in

varied range and catapults from \$2.50 to \$100.00 per hydrant per year in various water utilities. The total value of plant devoted to fire protection service, size of the community or area served, adequacy of the fire protection given, or operation, maintenance and depreciation expenses involved apparently have little influence, ordinarily, in determining this charge. This is not to say that water utilities generally are

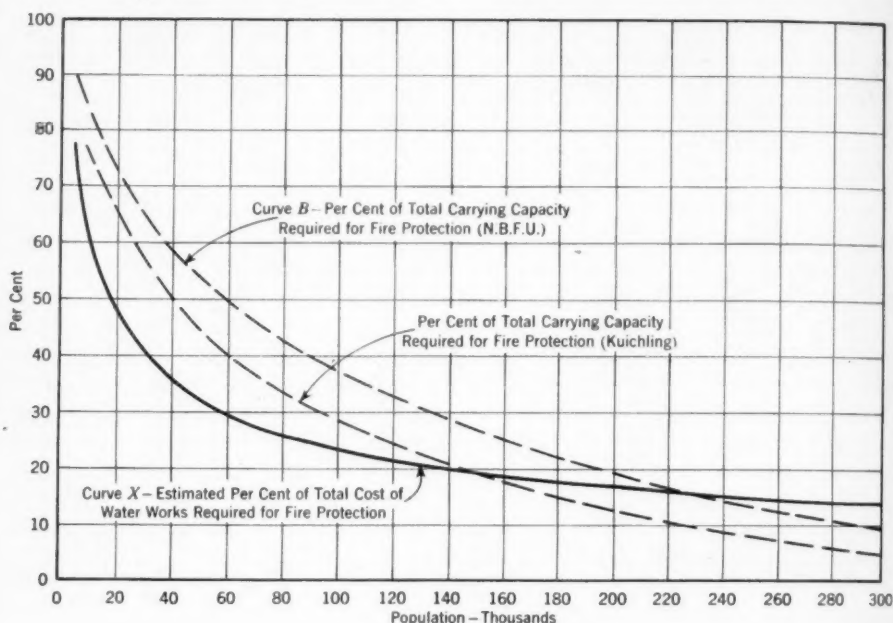


FIG. 1. Fire Protection Cost Data

Indiana, was asked by the public counselor how the annual hydrant charge was fixed. The answer was: that it did not represent the rate that should be earned, based on the facilities installed to provide the service; that there was no definite formula for arriving at the fire hydrant rental; but that it was mostly a matter of negotiation.

Reports and tariffs on file with the commission reveal that the annual charge per hydrant has a wide and

not earning an equitable return on their total plant value, but that the revenue from that portion devoted to fire protection is often deficient.

Where fire protection charges are set up separately in the utility's rate schedule by some arbitrary or get-together agreement, the income secured is merely supplementary to revenue obtained from general customers, and is not a fair accounting to them of how total revenue is obtained.

Value of Plant Required

If the basic formula which is generally followed by regulatory bodies—to consider an equitable rate of return on the fair value of the plant used in providing service—is accepted, a common starting point will be established. The first step to be taken, then, is the separation of that portion of utility plant used in fire protection from that used in providing general consumer service. This is no easy task. Certain items of plant, such as hydrants, connections and some valves, are used exclusively for fire service; some items, such as customer meters, services and filtration equipment, are eliminated; and other major items, such as pumping equipment, reservoirs and standpipes, and distribution and transmission mains, are used and allocable jointly. Contributions in aid of construction, or all property and plant donated by water users to obtain service, should be deducted before any division is made between consumer and fire protection service. A certain surplus capacity that is generally maintained for both, which may be expressed as a surplus or reserve capacity, should be divided between both services.

Probably the best-known simple method of determining that portion of a water works system devoted to fire protection is the result of a research study conducted in 1910 by Metcalf, Kuichling and Hawley for the Pennsylvania Water Co. (1). They devised a graph (curve X, Fig. 1) and a formula to apply in places of varying population. This formula is:

$$Y = \frac{147}{X^{0.31}} - 12.1$$

in which Y equals the per cent of plant and X equals the population in thousands. Thus, the percentage of plant

devoted to fire protection service varies somewhat as shown in Table 1.

These engineers intended the formula and curve to be of comparative value only, indicating the general trend, but unreliable if applied to individual plants where there is much deviation from normal.

In the smaller cities of about 10,000 population and less, the author believes it advisable to factor or adjust this formula by the deficiency points of the particular water utility, as reported by the fire underwriters. In the National Board of Fire Underwriters formula, with 1,700 points representing the ideal

TABLE 1

Portion of Plant Devoted to Fire Protection

Population	Part of Plant per cent
2,500 to 5,000	77 to 98
5,000	77
10,000	60
20,000	46
40,000	35
60,000	29
80,000	26
100,000	23
200,000	16
300,000	13

plant, meeting all requirements, this factor or multiplier is:

$$\frac{1,700 - \text{deficiency points}}{1,700}$$

and is applied to the percentage obtained in using the Metcalf, Kuichling and Hawley formula. For example, the proportion of plant used in fire protection for a utility rated to have 560 deficiency points, in a city of 10,000 population, is:

$$\frac{1,700 - 560}{1,700} = 67 \times 60 = 40 \text{ per cent}$$

It may perhaps be helpful here to cite some opinions of others relating to the general data prepared by Metcalf,

Kuichling and Hawley. One state public utility commission reported in 1937 that the parts of total plant apportioned to fire protection service in the state ranged from 11 to 66 per cent, depending on the size of the community and the equipment installed. A utility commission engineer of an eastern seaboard state reported in 1946 that, on the basis of his thirteen years of experience, he would not use figures greater than 60 per cent for cities and towns of 10,000 or less. Engineer Metcalf, co-author of the formula, gave testimony in a 1923 rate case involving a mid-western city of 300,000 population that 18.5 per cent of the total fair value of the water

cent in communities of about 50,000; 20 to 30 per cent in communities of about 100,000 population; and 10 to 20 per cent in the largest cities.

Maximum Demand Method

Another method that may be given some consideration in determining portion of total plant allocable to fire service, is sometimes known as the "maximum demand method." It is computed on the basis of the ratio of the maximum fire demand flows to the total combined demand for ordinary fire and general consumer service, according to the National Board of Fire Underwriters formulas:

Maximum Fire Demand

$$\frac{P}{5} + 10 = \text{no. fire streams} \times 200 \text{ gpm.} = \text{gpm. required}$$

Minimum Fire Demand Required

$$1.7\sqrt{P} + 0.03P = \text{no. fire streams} \times 200 \text{ gpm.} = \text{gpm. required}$$

General Service

$$2 \times \text{max. day pumpage in gpm.} + 500 \text{ gpm.} = \text{gpm. required.}$$

The term P represents population in thousands.

An example of the application of these formulas may be given. In a city of 10,000 population, with the maximum daily pumpage rate amounting to 900 gpm., the results would be:

	<i>gpm.</i>	<i>per cent</i>
Maximum fire demand	2,400	41
Minimum fire demand	1,200	20
General service	2,300	39

$$\frac{\text{Max. fire demand}}{\text{Min. fire plus general demand}} = \frac{41}{59} = 69 \text{ per cent.}$$

utility plant was devoted to fire protection service. In the A.W.W.A. *Water Works Practice Manual* (2), it is stated that experience has shown the cost of the portion of the water works involved by fire protection service to constitute from 60 to 80 per cent of the entire cost of the physical property for communities of less than 10,000; 30 to 40 per

Caution should be exercised in using this method, for local conditions may require that other factors be given weight. It should be borne in mind that, although the standards of the National Board of Fire Underwriters are admirable and desirable, and a nearer approach is constantly being made toward them, many communities have

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found it financially impracticable to attain so liberal a standard as they prescribe.

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An actual application of the principle may be found in a recent hearing before the Indiana Public Service Commission. The water utility in City A, of 3,900 population, petitioned for an increase in general customer rates, alleging insufficient revenue to meet operating expenses and pay a reasonable return on the fair value of property and plant. An engineering analysis was made of the utility's operations, as it was apparent that fire protection charges were unusually low. The portion of the plant devoted to fire service, according to the Metcalf, Kuichling and Hawley curve, was 85 per cent; the fire underwriters listed 734 deficiency points. On this basis, the proportion of total plant devoted to fire protection was:

$$\frac{1,700 - 734}{1,700} \times 85 = 48 \text{ per cent}$$

city of
0 gpm.

The portion devoted to fire protection service was likewise computed according to the "maximum demand" theory, the calculations by this method being tempered or factored further as it was thought local peculiarities pertaining to City A warranted, and a resultant of 42 per cent was obtained.

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The average of the two latter methods, or 45 per cent, was determined to be the portion of total plant devoted to fire protection service. By an accountant's allocation between general service and fire protection of City A's operating expenses, the commission determined that this utility's revenue from the 55 per cent of the plant used in general consumer service was adequate, but that the revenue from the 45 per cent devoted to fire service was greatly deficient. Increase in customer rates

was denied, and authority granted to raise fire protection charges to the proper level.

After that portion of water utility plant devoted to fire service is established, an equitable return, after operating expenses are deducted, would be that per cent of the fair value of property and plant determined to be fair and reasonable. An allocation of total operating expenses between general service and fire protection would, of course, be necessary, but no exhaustive analysis of that will be attempted here. Some items of expense, such as purification, service, meter and hydrant maintenance, are directly chargeable; other items are allocable; and one basis of allocation might be capacity against output.

The mechanics of applying charges to realize the total revenue necessary to bring an equitable return on the fair value of property used in fire protection service should be non-discriminatory to the customers, the municipality and the utility alike.

Usage Charges

Although it has been generally opposed, at least by those who have written on the subject, there should be mentioned, purely for argument's sake, the simplified method of adjusting general customer rates to include fire protection charges allocated over all water customers. On this basis, customers would pay the additional charge proportioned to usage. Total plant revenue would be fixed on a fair value of total water utility plant, with no attempt made to separate the portion used for fire protection facilities.

The objection usually raised to this method is that the water customers bear the full expense of fire protection, and those property owners or residents

who are not water customers receive fire protection without cost. Customers in built-up business sections, known as congested value districts, where property values are high and fire protection needs greater, are held by some not to be large enough users of water to pay their just portion of fire protection. In Denver, Colo., in 1939, the greatest demand for fire protection service came from the congested value districts, whereas the greatest production of revenue was from the residential districts.

Proponents of this simple method point to the high ratio of water customers to total householders, which is constantly on the increase, and to the accelerated activity of the fire underwriters in congested value districts in demanding sprinkler systems and enlarged service connections, with resultant higher water usage and charges. Further detailed study on the part of plant managers and operators will no doubt be directed to the merits or faults of this simplified plan.

It is the more general and accepted practice to look to general funds appropriated by the municipality to compensate the utility for fire protection service. This is the fact which places both the municipality and the water utility in the category of bargaining agents. The utility's interests require carefully prepared figures to obtain the equitable charge warranted. Total charges are now very commonly applied on the basis of hydrant charges, or by dividing the total annual gross revenue anticipated by the number of hydrants in service, thus arriving at an annual charge per hydrant. Although this method is followed by most water utilities in Indiana, it is in many respects not desirable. It works to the disadvantages of the utility because city

officials may request long pipe runs with relatively few hydrants, producing insufficient revenue on investment required. It hinders the securing of adequate fire protection facilities, for the city officials are inclined to limit the number of hydrants and save tax funds. It lends more incentive to bargaining when adjustments in rental charges are made. Overemphasis is given by the layman to the charge per hydrant, and too little attention is given to the various accessory equipment required to render adequate protection.

Equitable Charges

The charge per inch-foot of pipe distribution system, with a nominal annual allowance per hydrant for maintenance, is generally considered the most equitable of the methods yet devised. This system, or one very similar to it, is followed by only four or five water utilities in Indiana today. The total fire protection revenue anticipated is apportioned in part in the form of a nominal annual charge of from \$10 to \$15 per hydrant for maintenance. The balance is apportioned over the distribution mains of 6 in. diameter and larger as an annual charge per inch-foot, that is, the product of the inside diameter of the pipe in inches multiplied by the total length in feet. In smaller plants, where hydrants are installed on 4-in. mains, it would be reasonable to compute main charges on 4-in. and larger pipe. The advantages of this method of applying fire protection charges are readily apparent. As extensions to plant are made, proper charges to maintain a fair and equitable return on all plants used in service are adjusted automatically. Where municipal officials request hydrants in relatively undeveloped areas, adequate revenue to pay a fair return on utility's

plant investment is immediately forthcoming. Installation of adequate fire protection facilities in the community would more likely be accelerated for the benefit of all concerned.

It is well to point out that a further detailed study on the part of all plant managers and operators in the matter of public fire protection charges is highly desirable. The old idea that the best to be expected was reimbursement for actual cost, or perhaps as small a net loss as could be managed, is detrimental to the safety and health of the community. The fire underwriters are doing a notable job in alleviating dan-

gerous fire hazards, and the urgent necessity for their work has filled the front pages of newspapers, particularly in the year just passed. Water utility operators and the general public as well must lend every aid to curtailing such losses.

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Finding the Low Point on the Cost Curve

By Warren E. Howland

A paper presented on May 9, 1947, at the Indiana Section Meeting, Indianapolis, Ind., by Warren E. Howland, Prof. of San. Eng., School of Civ. Eng., Purdue Univ., West Lafayette, Ind.

EVERYONE is familiar with the definition of engineering as the economic utilization of the materials and forces of nature for the benefit of man. And by the word "economic" is usually meant "doing for one dollar what any bungler could do for two, after a fashion." But real economy means doing with 98 cents what an otherwise intelligent engineer would do for a dollar. Such a saving requires taking advantage of the last possible economy—without, however, in any way reducing the safety or reliability of the structure, or cutting down on capacity, quality or any other desirable operating or construction characteristics. It means doing a particular job strictly in accord with the best accepted standards of good engineering practice, and, at the same time, choosing such proportions, such sizes, lines or grades as involve the very least apparent total annual cost.

The costs to be avoided are what a famous old engineering author, A. M. Wellington, has described (1) as those "operating expenses from bad location [and design] that come by a gentle but unceasing ooze from every pore . . . resulting in a loss vastly larger than any possible loss from bad construction."

The economy proposed is not dangerous nor radical in any way, nor is it spectacular. It means saving a little here and a little there on material or

labor at the cost of more painstaking, careful, prolonged thinking by the designing engineer. Nor will the extra cost for the engineer offset the savings he effects. He would have to be paid much more than he is before the economies effected would be dissipated in fees.

A few examples may illustrate this concept and make it clearer:

The Four-Tenths Rule

In 1907 a paper was written (2) on what was called the four-tenths law for pipe design. A. L. Adams, a practicing engineer, showed that, by an actual analysis of cost data, it was possible to obtain the low point in the cost curve for the design of a water pipeline. The method used was to so proportion pipe size and pump head that four-tenths (or two-fifths) of the annual charge for interest and depreciation on the pipeline would equal the annual cost of loss of power in pumping. In discussion of the paper, the rule was derived by purely theoretical considerations.

It is not at all difficult to show that this rule follows as a logical consequence of two conditions: the law that, other things being equal, the head loss through a pipe varies inversely with the fifth power of the diameter, and the assumption that the first cost of the pipe varies directly with the square of the diameter. If it appears that the head

loss varies with a somewhat different power of the diameter, say n , and that the cost of the pipe varies directly with the m th power of the diameter, then the rule becomes the m/n th rule. That is, the annual cost of lost power in the pipe should equal m/n times the annual charges for interest and depreciation. In this symbolic form, the rule is capable of general application.

and sewers, using data kindly made available to him by the engineering firm of Greeley & Hansen, for which he worked two summers ago. It was found that the cost of tunnels may vary with the first power of the diameter, of force mains with the 1.64 power of the diameter and of sewers with 9 and 6 ft. of cover with the 1.33 and 1.54 power respectively. Thus it would appear that the ratio m/n might vary from 0.20 for tunnels through 0.25 or 0.3 for sewers to 0.33 or even to 0.4 for water pipes—this being the ratio of the annual cost of the lost power (the smaller quantity) to the annual charge for interest and depreciation against the pipe.

Henry King of the Sanitary District of Chicago has used the same analysis and arrived at formulas which imply the same rule for the loss of head in air lines leading to the aeration tanks of an activated sludge plant (3).

It is to be noted in Fig. 1 that the low point of the total cost curve does not occur where the two separate cost curves cross (that is, where they are equal), but rather at a point where the diameter is 14 per cent higher than it would be at this point. As a result of using the larger diameter, the total annual cost is reduced 10 per cent below what it would be if the two items of cost were made equal to one another as at the so-called cross-over point.

Now there are problems in the economics of engineering where the cross-over point occurs at the minimum point on the cost curve. Such a problem is the design of a wire, for which Kelvin's law applies. This law states that the most economical wire will be obtained when the annual cost of lost power in the wire is just equal to the annual charges for interest and depreciation against the wire.

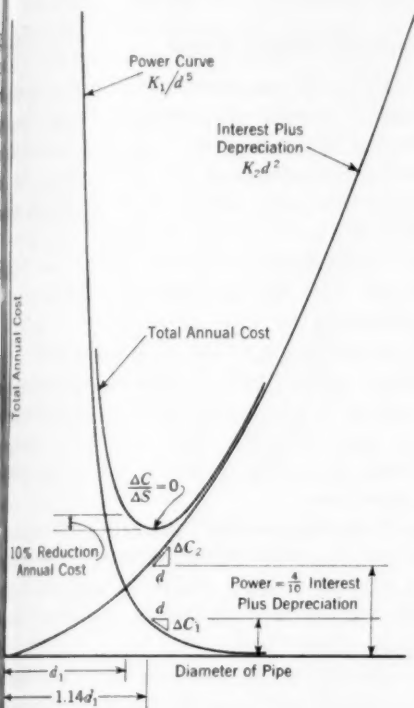


FIG. 1. Determination of Low Point in Pipeline Cost Curve

Using this rule, design formulas can be derived. When expressed in terms of diameter they often show that the head loss diameter is the $\frac{1}{3}$ root of certain other quantities. This $\frac{1}{3}$ root follows from the sum of m and n in the formulas being 3 when m is 2 and n is 5.

Some studies were made by the author of the costs of tunnels, force mains

This raises the question: "What is it, precisely, that determines the low point on the cost curve?" As can be plainly seen from these two examples, it is not always a particular equality or ratio between the various items of cost themselves. Instead, the significant relationship at the low point of the cost curve is the manner in which the various separate items of cost change at

decrease in the power costs, ΔC_1 , and thus the total cost does not change. In other words, there is at this minimum point a flat place in the curve, and we can say that a small change in the cost at this point, $\Delta C = \Delta C_1 - \Delta C_2$, divided by the corresponding small change in value of the diameter, Δd , is equal to zero. This ratio of change, $\Delta C / \Delta d = 0$, is the condition for a low point on the cost curve.

Rule for Equal Slope

There is another rule for the economic design of conduits that lies buried in the literature and probably is not taken advantage of as it should be. It is this: that along any single pipeline, the value of $\Delta c / \Delta s$ for every pipe in series along that path should be the same. In this formulation Δc is the difference in unit costs of two pipes (one the same as the pipe used and the other of slightly different diameter) and Δs is the corresponding difference in head loss per unit length of pipe. Both pipes are assumed to carry the same flow.

Sometimes called the rule for equal slope, because of the manner in which it may be applied, this rule was used in the design of the Catskill Aqueduct for New York City.

In Fig. 2, the cost per foot of length is plotted for three kinds of sections—a tunnel section (most expensive), a cut-and-cover section and an open channel section (least expensive). It is assumed that a pipeline consists of fixed lengths of each of these kinds of section. The question propounded is "What should be the slope of each section for best economy?"

The method requires that such a value of the slope of the hydraulic grade line shall be used that the slope of the tangent to each of the cost curves

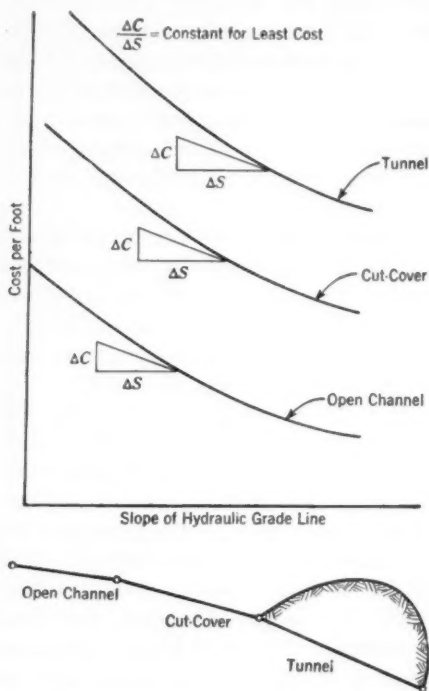


FIG. 2. Determination of Most Economic Slope for Various Sections

this point. It is a peculiarity of the low point on the cost curve that here small changes (in this problem small changes in the diameter) result in such increases in certain cost items as perfectly balance the resulting decreases in the other cost items. As the diameter increases, the resulting increase in interest and depreciation charges, ΔC_2 , is just exactly balanced by the resultant

the points selected shall be the same; that is, $\Delta c/\Delta s$ must be the same for all sections. As can be seen from Fig. 2, the open channel becomes the flatter section and the tunnel the steeper section in this example.

An interesting application of this rule results in the solution of the problem of locating the best point to change over from one size to another in a pipeline crossing a valley so deep that pressure increases become large enough to require significant increases of pipe thickness and decreases of pipe diameter (Fig. 3). The rule then requires a change to the next smaller size (in

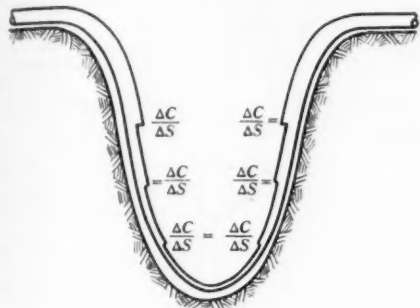


FIG. 3. Economic Determination of Change in Pipeline Size

proceeding downhill) or to the next larger size (in proceeding uphill) so that $\Delta c/\Delta s$ is everywhere the same at these change-over points. The difference in the cost per foot of adjacent sizes of pipe, Δc , divided by the corresponding differences in slope of the hydraulic grade lines of these two sizes of pipe, Δs , will then be the same at each change-over point. This principle was worked out by the engineers for the Los Angeles Aqueduct.

The simple rule just given for design of a water conduit has been extended by the author to sewer design, in which excavation plays an important role in

selection of grade (4). In sewer design $\Delta c/\Delta s$ should not be the same throughout the length of the sewer. Instead, the difference between the sum of these quantities, $\Sigma \Delta c/\Delta s$, for the upstream sections joining at a manhole, should equal the value of $\Delta c/\Delta s$ for the downstream section plus the value of one-half the cost per foot of depth of all the sewer trenches joining at this manhole. An interesting consequence of this rule is that the sewer becomes progressively steeper downstream.

Professor Camp (5) has made a very complete analysis of the economics of design of pipe grid systems using essentially the two rules for design just discussed, and has prepared a number of

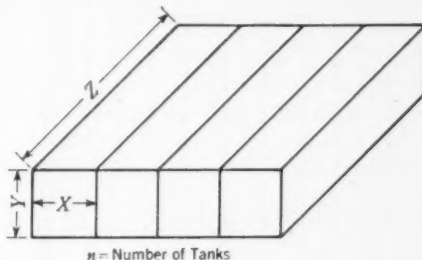


FIG. 4. Most Economic Proportions of Rectangular Tanks

rather involved formulas which are worthy of consideration. These complications, however, have purposely been avoided in this discussion, in order to emphasize the simplicity of the concepts and of certain of the rules for design instead.

These rules may be summarized as:

1. The so-called 4/10ths rule or, better, the m/n th rule for the design of water pipes through which water is pumped
2. The rule for equal values of $\Delta c/\Delta s$ in conduit design
3. The extension of Rule 2 to cover the design of open channels where ex-

cavation is influenced by the elevation of hydraulic grade line, as in sewer design.

Tank Design

As a final group of problems to illustrate some of the rules available for economic design of water works structures, a few simple considerations for the design of tanks may be presented. In these examples it will be assumed

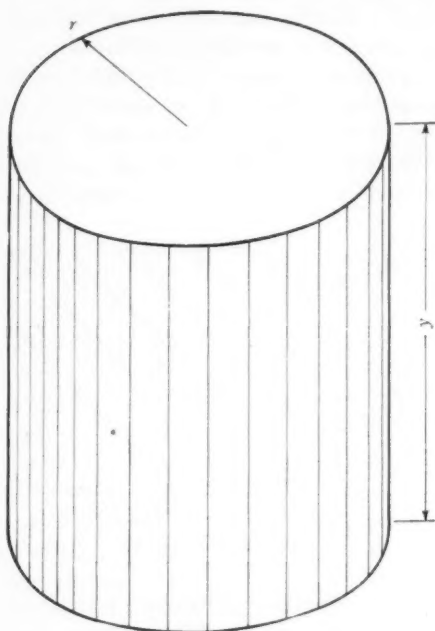


FIG. 5. Most Economic Proportions of Cylindrical Tank

that the volume is fixed and it is desired to obtain the least cost of installation of tank for the given volume. Actually there are many problems of design of tanks—of sedimentation tanks especially—where volume is of little importance* regardless of engineering

*In settling tanks, the design should be based upon a given rate of application of fluid per square foot of horizontal surface area of tank, rather than upon the usual given volume for a given time of retention.

opinion to the contrary, and for these, therefore, the rules being presented are of little if any importance. Nevertheless, storage tanks and coagulation tanks should be designed for a given volume, and the rules may be of some value in their design:

1. It is desired to determine the most economic proportions (width x , length z , height y) of a number n of rectangular tanks having $n-1$ common walls (Fig. 4). The relative costs per square foot of bottom, interior walls, exterior walls and top are $1:k:l:m$, respectively, and the fixed volume of each tank, V , is known. The answer to this problem is given by:

$$x = y \left[\frac{2l + k(n-1)}{n(1+m)} \right] \quad z = \frac{2yl}{1+m}$$

Note when $k = l = m = 1$ and $n = 1$, then:

$$x = \frac{1}{1} \left(\frac{2}{1+1} \right) y = y$$

and:

$$z = y \left(\frac{2}{2} \right) = y.$$

That is, $x = y = z$, and the tank becomes a cube (which it should be). If this single tank has no cover, then $m = 0$; and if $k = l = 1$, then:

$$x = y \left(\frac{2}{1} \right) \text{ and } z = 2y.$$

That is, $x = z = 2y$. The shape is a half-cube (which it should be).

The cube is a shape in which a sphere can be inscribed and the half cube is a shape in which a hemisphere can be inscribed. This suggests that the shape of minimum surface for a given volume or the shape of maximum volume for a given surface is, when covered, one in which a sphere can be inscribed and, when uncovered, is one in which a hemisphere can be inscribed.

Circular tanks will not nest like rectangular tanks and so cannot take advantage of the saving that comes from the sharing of walls by adjacent tanks; hence they will be considered singly. As single tanks they are more economical in construction than are single rectangular tanks. Also, the scraping mechanisms that are placed in circular settling tanks are of simple design and construction. For these reasons, circular tanks are often used.

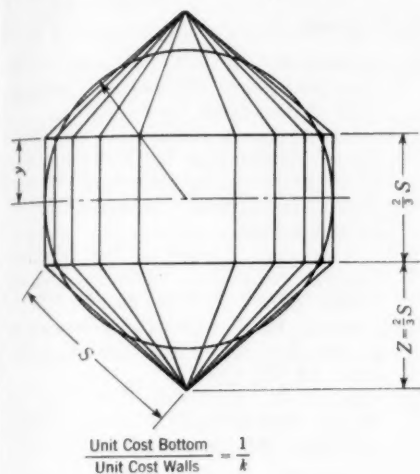


FIG. 6. Most Economic Proportions of Conical Bottom Tank $r = \frac{\sqrt{5}}{3} S$

It is desired to determine the most economical proportions of radius r and height y for a single circular cylindrical tank (Fig. 5). The relative costs per square foot of bottom, walls, and top are $1:k:l$, respectively, and the fixed volume V of the tank is known. The answer to this problem is:

$$y = r \left(\frac{1+l}{k} \right)$$

It is to be noted, when $1 = k = l$, that $y = 2r$, or the height of the cylinder is equal to its diameter (as it should

be). Also, if $k = 1$ and $l = 0$, then $y = r$, which is the condition for a cylinder whose height is half its diameter (as it should be). These are shapes in which a sphere can be inscribed.

Another interesting problem arises when the bottom of the tank is made in the form of a cone of height z and the top is omitted. Such a tank is one-half of the volume shown in Fig. 6. It is desired to determine the most economic proportions of this tank. The relative costs per square foot of bottom and walls are respectively $1:k$. The answer to this problem is $z = \frac{2}{3} sk$;

$$y = s \left(\frac{1}{k} - \frac{2}{3} k \right)$$

and:

$$r = \sqrt{s^2 - z^2} = s \sqrt{1 - \frac{4}{9} k^2}$$

where s is the slant height of the cone. Note that when $k = 1$, $z = \frac{2}{3}s$, $y = \frac{1}{3}s$, and the radius of inscribed hemisphere is $\sqrt{\frac{5}{9}}s$, which it should be; that is, this is a shape in which a hemisphere can be inscribed.

Open Channel Problems

Several other hydraulic or sanitary engineering design problems may be listed for which simple rules for economic design are available.

1. Determination of the best trapezoidal shape for an open channel of which either lining or excavation is the main factor in the cost.

Answer: The one in which a semi-circle can be inscribed, or the one whose hydraulic radius is half the depth.

2. Determination of the best slope for a drainage ditch along the land side of a river levee designed to carry a fixed flow to drain a swamp below river

level, the canal to enter the river at a downstream point.

Answer: Slope of canal = $\frac{2}{3}$ slope of river.

3. Determination of the best slope for a power canal required to parallel a river of constant slope and develop a certain power by allowing water to drop through turbines from canal to river at a downstream point.

Answer: Slope of canal = $\frac{2}{3}$ slope of river.

The answers to problems 2 and 3 depend somewhat on the formula used.

Of course, it is not possible to arrive at the best economical solution to a problem by the mere application of simple rules like the ones just presented. In the hands of experienced or trained persons, however, these simple rules and formulas can be very helpful suggestions and guides, and that is all the author claims for them. Admittedly actual cost studies for particular and peculiar engineering problems are needed. Plan *A*, Plan *B*, Plan *C*, and maybe even more plans should be studied to find the one of the very least

possible cost. The author hopes that water works executives will make or have made a lot of plans—not merely in order to employ more of his students, but because he thinks that multiple engineering studies will really save a great deal in the long run, and therefore more such engineering studies should be made. In making these plans, particularly the preliminary plan, such rules as have been mentioned will often prove helpful.

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Economics of Water Storage

By Norman G. McDonald

A paper presented on Apr. 15, 1947, at the Canadian Section Meeting, Montreal, Que., by Norman G. McDonald, Partner, Gore & Storrie, Cons. Engrs., Toronto, Ont.

RESERVOIRS are considered to be economical when their use reduces the total over-all annual cost of water system operation, including interest on investment, depreciation and operating charges. The consideration of economy in a municipal water works system should not be limited to the system alone but generally should include the community as a whole. For example, provision of water storage may permit pumping during off-peak periods of municipal power load, when the discount to the water works, under the usual power contracts, is 25 per cent. Although this discount does not ordinarily save the water customer nearly as much as the equalized power load saves the power user, the total annual saving in power is often greater than the annual cost of the reservoir. It is not recommended that water customers be asked to make large expenditures to profit the electric customers, but, as all citizens are users of both water and electricity, the saving to the whole community should be considered.

That the broader view of economy in municipal water works is justified is well illustrated by an experience at Oshawa, Ont., a few years ago. One very cold winter night the water works intake was clogged with ice, and, by 6:00 A.M., with practically no water coming through, the storage was be-

coming depleted. All the factories in this highly industrialized city, as well as schools and other large water-consuming establishments, had to be closed. The direct loss to the water works department was only that resulting from the loss of water sales, amounting to about \$150. The actual loss to the city as a whole is difficult to estimate, but it was probably thousands of dollars—equal to the cost of constructing a 1-mil.gal.* reservoir which would have enabled the water works system to maintain the supply.

There are three types of reservoirs used by water works: the impounding reservoir, formed by the construction of a dam across a stream; the low-level reservoir, such as is commonly used at water purification plants or on well supplies; and the high-level reservoir, which is constructed at a sufficiently high elevation to provide adequate operating pressure in the distribution system.

The primary purpose of all reservoirs is to store water when the supply exceeds the demand and to release it when conditions are reversed. When storage is available, the required capacity of the supply lies between the average and the maximum demand, depending upon the amount of storage provided.

* Imperial gallons are used throughout.

Impounding Reservoirs

Impounding reservoirs are used chiefly where the stream flows at certain periods of the year are inadequate to meet the demands. Some or all of the water available during the periods of high flow is therefore stored for use during the periods of drought.

Another important function of this type of reservoir is to provide a depth sufficient to obtain continuous withdrawal of an adequate supply of water. The shallow stream presents a troublesome situation for the withdrawal of water because of the difficulties encountered from ice and silt.

An impounding reservoir may be of great flood control value on a stream by storing a large part of the flood water and allowing it to discharge at a rate which will not damage the property downstream. Hydraulic power for pumping the water or for generating electricity may also be obtained through the construction of a dam and impounding reservoir of sufficient size.

One of the best Canadian examples of an impounding reservoir is the Glenmore Reservoir of the Calgary (Alta.) Water Works System, which was formed by the construction of a dam across the Elbow River. It has a storage capacity of 4,870 mil.gal., of which 50 per cent is in the upper 11.3 ft. An additional 1,360 mil.gal. storage may be made available through the installation of flash boards 5 ft. high on the crest of the dam spillway.

The flow in the Elbow River varies over a wide range. The average flow is about 220 mgd., the minimum 10 mgd., and the maximum 14,000 mgd. The minimum flow always occurs during the very cold weather; the maximum or flood flow usually occurs in the early part of June. The construction of the reservoir insures a continu-

ous water supply of over 80 mgd. during as dry a year as has been recorded. As the present water consumption averages only 20 mgd., or 30 mgd. on the maximum day, water is available to generate hydraulic power and, during the fourteen years that this plant has been in operation, all pumping has been done by hydraulic power except during a 19-week period in which one of the electric motor driven stand-by pumps had to be operated.

Before the construction of the Glenmore Dam, extensive damage was done in the city by the flooding of the Elbow River. In the construction of the dam, two flood gates were installed, each 12 ft. in diameter. These gates have a combined capacity of about 9,000 cfs., which is all the river channel will carry without causing damage by flooding. By maintaining low water in the reservoir during flood season and operating these gates properly, the intensity of the flood can be materially reduced. This was well illustrated in June 1932, before the project was entirely completed, when the flow in the river entering the reservoir reached a peak of 26,000 cfs., but the flow downstream did not exceed 12,000 cfs. This flood was 70 per cent higher than any previous flow recorded, and, if uncontrolled, would have caused very severe property damage in the Elbow Park residential section of the city.

The flood gates are also very useful in preventing the deposition of silt in the reservoir. The opening of the gates and the lowering of the water level during high flows in the river allows the silt-laden water to pass through. When the flow subsides, the reservoir is filled with clean water. The operation of the flood control system does not interfere with the normal water supply to the city.

The proper evaluation of the various features of this reservoir is somewhat difficult. The bridge over the top of the dam provides access to the area on the opposite side of the river but what its actual value may be cannot be definitely determined. Similarly, although flood-caused property damage, which would have been very heavy without the protection of the reservoir, has been practically eliminated, the resultant saving is difficult to evaluate. The value of the water power can be estimated with a reasonable degree of accuracy; it amounts to about \$45,000 per year. On an original investment of \$1,100,-

the distribution systems to provide the necessary pressure are the most desirable. Where there is suitable high ground close to the distribution system, the best type of reservoir is one constructed of reinforced concrete at ground level, roofed over and covered with earth. This type of reservoir is low in cost and requires very little maintenance.

Where the ground is not quite high enough to provide the required pressure, a standpipe may be used. This type of reservoir is usually of steel constructed on top of the ground, although reinforced concrete may also be used. The concrete must, however, be of high quality to withstand the weathering action, particularly in cold climates. In standpipes only the top 25 ft. should be considered as effective storage capacity, although the lower water is also of some value.

Where the height of the reservoir above the ground exceeds about 60 ft., it is generally economical to use an elevated tank in which all the stored water is useful. The elevated tank is nearly always constructed of steel, although a few tanks have been built in reinforced concrete, mostly of the prestressed design.

Elevated tanks should have a total depth of about 25 ft. of water; their diameters vary with the capacities required. Small tanks are made with hemispherical bottoms whereas larger tanks with capacities of from 100,000 to 500,000 gal. are made with ellipsoidal bottoms and large riser pipes. Tanks with capacities of 500,000 gal. or larger are now being made with radial cone bottoms to maintain the normal depth with increased diameter.

As an elevated tank occupies a prominent place in a community's landscape, the older tank designs, which

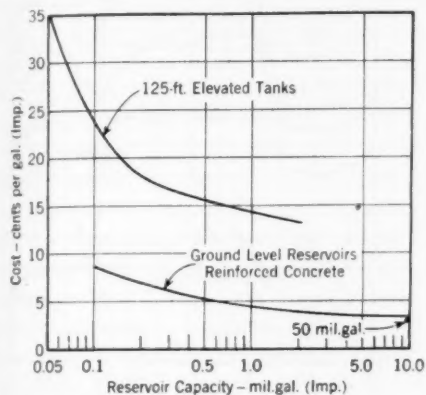


FIG. 1. Capacity Cost of Reservoirs

000, this represents a return of over 4 per cent. Taking into consideration the various services rendered by this reservoir, its use as a part of the water works system is undoubtedly economical.

Service Reservoirs

Reservoirs for the storing of water before distribution are very useful in water works systems and frequently are responsible for substantial savings in the capital investment and operating costs. High-level reservoirs constructed at sufficient elevations above

were very unattractive in appearance, generally had to be built in out-of-the-way places, where the costs of the long connecting pipes sometimes made them uneconomical. In recent years the general appearance of tanks has been receiving more consideration, and better designs are now available. One of the first modern designs was installed at Perth, Ont., about ten years ago, and was a decided improvement on the old conical roof type.

The large tanks with ellipsoidal roofs, radial cone bottoms and tubular columns, such as those built at Scarborough, Ont., last year, and now to be constructed at Sarnia and Sudbury, Ont., are greatly improved in appearance and can be located in almost any area without greatly reducing the value of the surrounding property.

Reservoir Location

The most desirable location for a storage reservoir from the standpoint of distribution is within the area served by the system, near the side opposite the source of supply. Such a location makes possible a short main connection, minimum pressure variations and feeder mains of economical size. It also provides greater reliability of supply, for, if there is a main break, only the section between valves need be without water; the upper side can be fed from the reservoir, and the lower side from the source of supply. Although a reservoir should generally be constructed on high ground, a study of the topography, distribution main sizes and other factors should be made to determine the most economical location. The connecting main should be considered as part of the reservoir cost; it is not economical to go very far to locate an elevated tank on high ground, because the cost of the connecting

main may exceed the saving effected by decreasing the height of the supporting columns and riser pipe.

Connecting mains should be of liberal capacity to minimize pressure variations. In general, the connecting main need only carry the difference between the rate of pumping and the demand, which may be as little as about 50 per cent of the average demand. If used for fire service, the outflow rate will be materially increased, and if pumping is stopped during electric

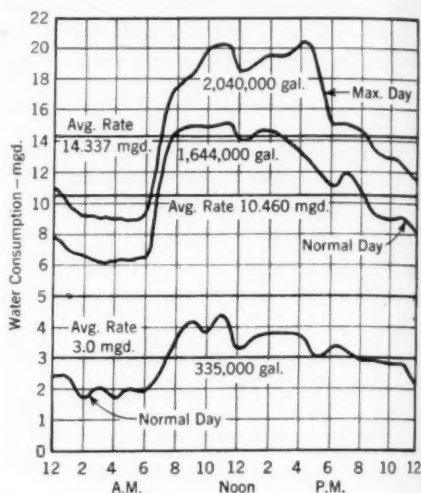


FIG. 2. Typical Variations in Consumption

peak load periods, the capacity of the connecting main should be about twice the yearly average demand. With all other conditions similar, long connecting mains should be larger than short ones to obtain the same total loss of head.

Reservoir Costs

Reservoir costs have greatly increased during the past few years, and are now from 60 to 90 per cent higher than they were ten years ago. As lumber and other materials become more

readily obtainable and labor more efficient, lower costs of concrete construction are to be expected.

Construction costs vary in different municipalities and are affected by local conditions, but the approximate costs of various sizes and types of reservoirs have been shown in graphical form in Fig. 1. These curves show the cost per gallon of storage capacity for elevated steel tanks 125 ft. high to the top water level and for reinforced concrete reservoirs constructed at ground level and covered and embanked with earth. The depth of water is 25 ft. for the elevated tanks and varies from 23 ft. for the largest concrete reservoirs to 8 ft. for the smallest. Excavation is assumed to be of earth. Foundations, but not the connecting mains, are included in costs.

The curves indicate that the unit costs increase rapidly with the reduction in size of reservoir, particularly with the elevated tank. For this reason, the use of elevated storage in small water works systems is generally not economical, and it is cheaper to pump continuously by electric power, using a gasoline engine auxiliary.

Amount of Storage

The provision of high-level storage is justified in all but the smallest water works systems to maintain supply during periods of power failure until stand-by pumps can be put into operation. The use of reservoirs for this purpose will in general only be considered economical if the effect on public relations is evaluated. For this purpose, a 50,000-gal. elevated tank is valuable even in cities of moderate size, although larger tank sizes are desirable.

The amount of storage required for balancing—that is, for absorbing the variations in the demand above and

below the average—varies in different municipalities. The effect of the supplying of water to industries on the amount of storage required depends a great deal on the type of industry and, particularly, whether it is operated on an 8-hour or 24-hour basis. The curves in Fig. 2 show the consumption in two municipalities. The lowest curve shows a normal day's consumption for the city of Oshawa, Ont., which has a fairly heavy industrial consumption. The storage capacity required for balancing the demand is 335,000 gal. representing 11 per cent of the day's consumption. For the maximum day, a capacity of 750,000 gal. is required—about 18.5 per cent of the maximum, or 25 per cent of the average, day's consumption.

The upper curves of Fig. 2 show the variation in the water consumption for the normal and maximum days for district No. 3 of the water works system of the city of Toronto, Ont. For the normal day the storage capacity that would be required for balancing is 1.644 mil.gal., which is 15.7 per cent of the day's consumption. The upper curve indicates a storage requirement of 2.040 mil.gal., which is 14.2 per cent of the average consumption on the maximum day, or 18.5 per cent of the consumption on the average day.

An examination of the water consumption records of a number of municipalities indicates that, in general, a balancing storage capacity equal to one-quarter of the consumption of the average day of the year, or one-fifth of the consumption of the average day of the maximum month is required. This amount will permit the supply equipment to be operated at the average consumption rate for any day.

For operation of the pumping equipment during off-peak periods of the municipality's electric power load, suf-

ficient storage should be provided to supply the demand of the maximum hour, or about 10 per cent of the average day's demand.

The economical reservoir capacity to be provided for any water works system depends on local conditions and can only be determined by making a thorough study. Assuming that the water works system has to provide fire service, its capacity must be sufficient to meet the fire service demand for a period of 10 hours in addition to the domestic requirements. When estimating domestic demand for fire service calculations, the average of the maximum

domestic demand, to supply part of the water for fire service and to provide for the operation of pumps in the off-peak power load period, may be determined from the formula:

$$R = aD + bD + \frac{10}{24}(D + F - S)$$

in which R is the reservoir capacity in million gallons, D is the average domestic demand for the maximum month in million gallons per day, F is the fire service demand in million gallons per day, and S is the capacity of the supply or pumping equipment in million gallons per day. The terms a and b are

TABLE 1

Storage Reservoir Requirements for Various Supply Capacities

Population	D	F	$S = D + F$	$S = D + \frac{1}{2}F$	$S = D + \frac{1}{3}F$	$S = D + \frac{1}{4}F$
1,500	0.150	0.600	0.045	0.107	0.170	0.232
2,500	0.250	0.900	0.075	0.168	0.262	0.355
6,000	0.600	1.500	0.180	0.336	0.492	0.648
10,000	1.000	2.400	0.300	0.550	0.800	1.050
20,000	2.000	3.600	0.600	0.975	1.350	1.725
40,000	4.000	4.800	1.200	1.700	2.200	2.700
60,000	6.000	6.000	1.800	2.425	3.050	3.675
100,000	10.000	7.500	3.000	3.780	4.560	5.340

month's consumption, rather than the annual average, should generally be taken. This maximum is usually found to be in the month of July or August and may be from 10 to 50 per cent or more above the yearly average.

The period on which the domestic consumption should be based varies in different municipalities because of climatic conditions and the characteristics of the water demand. A period as short as three days may be justified in a community in which the water consumption increases greatly during periods of drought and high temperatures.

The amount of storage required in a reservoir to be used for balancing the

the fractional parts of the domestic demand required for balancing and off-peak operation, respectively.

Table 1 shows, for population of 1,500 to 100,000, the storage reservoir requirements for various supply capacities, using the fire service rates F specified by the fire underwriters and assuming an average domestic demand D for the maximum month of 100 gpd. per capita. The coefficient a is taken as $\frac{1}{2}$ and b as $\frac{1}{10}$. In addition to the reservoir capacities indicated, reserves for use during interruptions to the supply may be justified.

It is quite evident that if the supply plant is expensive—that is, if it con-

tains a water purification plant, low-lift and high-lift pumping stations and a long feeder main—a large reservoir is economical, particularly if it can be constructed of concrete on conveniently located high ground. If the supply plant is simple, however, and consists only of a pumping station and short feeder mains, the economical size of reservoir would be much smaller. If high ground is not available and an elevated tank has to be used, the economical size is greatly reduced.

If the source of supply is deep wells, or if a water purification plant is used, the provision of storage facilities is always economical. The capacities of many existing water works plants can be economically increased by the use of storage reservoirs.

Although high-level storage is desirable for the whole capacity, there are many municipalities in which high ground is not available and elevated tanks must be used. In such communities it is economical to provide low-level storage in concrete reservoirs at the plants and elevated tanks of moderate size in the distribution systems. The economical sizes of such tanks will depend upon the variations in the demand, the cost of power for pumping and the lengths of feeder and connecting mains.

Auxiliary Pumps

During recent years there has been a marked development in diesel type internal combustion engines, and sev-

eral engines are now available in speeds satisfactory for pump drives. If storage can be provided only by an elevated tank with a long connecting main, it may be more economical to forego it, and operate diesel engine driven pumps during peak load periods. The Oshawa Public Utilities Commission operates two diesel engine driven pumps about four hours each week to deliver the greater part of the water requirements during electric peak load periods, because the mains to the elevated tank are too small to provide sufficient pressure in the distribution system during these periods. The electric pumps are operated on off-peak power with a saving to the city as a whole of about \$7,000 per year in electric power. The annual cost of fuel and lubricating oil for the engines is only about \$350, to which fixed charges add \$1,500, making a total of \$1,850 per year. These pumps are also very useful for carrying peak loads in water demand, and frequently are operated for short periods to avoid starting an additional electric motor driven pump. They are also operated during failures in the electric power supply and for fire service, in conjunction with large gasoline engine driven stand-by pumps.

Reservoirs in municipal water works systems play an important part in the economical supply of water. Their value in promoting continuity of normal service during periods of unusual demands or breakdowns of equipment may be of even greater importance.

Planning Capacity for Elevated Storage Tanks

By Donald H. Maxwell

A paper presented on May 7, 1947, at the Indiana Section Meeting, Indianapolis, Ind., by Donald H. Maxwell, Cons. Engr., Akword, Burdick and Howson, Chicago, Ill.

HOW much capacity should be planned for an elevated storage tank is a question involving operating convenience, efficient service, a factor of safety, and economics. The amount of storage that can be provided in any given tank may be governed largely by cost. Present conditions of increased construction costs make it more than ever necessary to consider how much capacity in elevated storage can be justified.

The early water works plants serving flat areas operated without elevated storage, except for the nominal storage that was sometimes provided by the familiar standpipe, usually at the steam pumping station.

With the advent of electric pumping, elevated storage became desirable primarily to effect pumping economies, provide a reserve against power interruption and furnish better pressures during periods of heavy consumption. Elevated tanks are now in general use in the more progressive and up-to-date plants. In the state of Illinois, which has an estimated total of about 700 water works plants, there are approximately 425 elevated tanks and standpipes, about 405 of which are tanks. Of this number, 3 have a capacity of 1 mil.gal. or more.

The leading tank manufacturers have made a substantial contribution through

the development of tanks of small depth and large diameter, which will provide the desired storage within the proper limits of pressure. Another noteworthy contribution they have made is in tank design, overcoming the objections raised in some quarters to the unsightliness of many elevated steel tanks. This improvement in appearance has tended to reduce cost in certain localities by making it unnecessary to hide the tank from view with an expensive masonry envelope.

Functions of Tanks

The functions of the elevated tank are:

1. To supply from storage that part of the peak rates of demand which is above the day's average rate of pumping

2. Incidental to this, by reducing the pumping rate and by permitting operation of the pumps more nearly at their rated head and capacity, and consequently at best efficiency, to reduce the cost of pumping by minimizing the demand charge and cost of energy

3. By providing reserve storage floating on the system, to insure uninterrupted service during a possible interruption of power

4. By proper location, to increase the efficiency of the distribution system, thereby improving pressures

5. Incidentally to provide some reserve for fire protection at times other than during the comparatively few days of maximum domestic consumption, although as a matter of economy the tank is not designed primarily for fire protection.

To serve its purpose best, the elevated tank or tanks should usually be located at the side of the distribution

rates of flow through the mains are thus substantially reduced, and friction losses are correspondingly less. As a result, smaller mains than would otherwise be required are found adequate, and there is less variation in water pressure, which is kept within the narrowest practicable limits.

In one system, the feeder mains required to provide adequate pressures

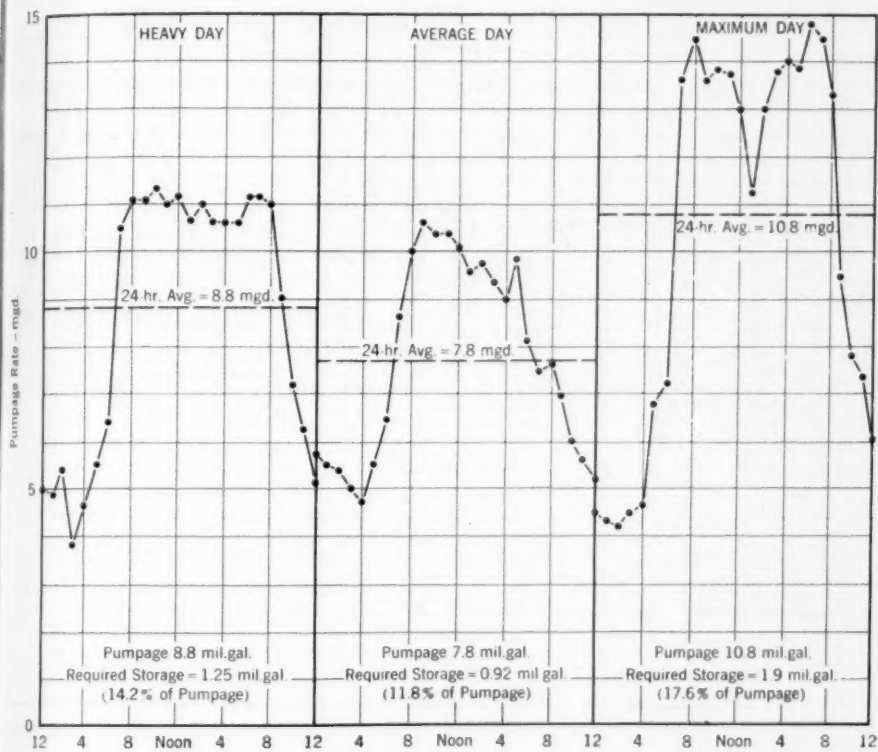


FIG. 1. Pumpage Variation and Required Storage—Plant A

system opposite to and remote from the pumping station. Thus, with the consumption centering in the area between the pumps and the elevated tank, the peak rate of demand is met partly from the pumping station and partly from elevated storage, the water coming from opposite directions. The

under conditions of maximum domestic consumption were estimated to cost 35 per cent more without an elevated tank than the mains required for comparable service with a tank at the far end of the area served. This difference, incidentally, offset about one-third of the cost of the tank.

Maximum Day Pumpage

In order to find the amount of elevated storage that is desirable, it is first necessary to determine the hourly pumpage rates on the day of maximum domestic consumption. If the pumping plant output is measured by a recording meter, this is a simple matter. Preferably, the maximum day should

Typical pumpage rates for an average day, a day of heavy pumpage and the maximum day for plant A, operating in a city of about 67,000 population, are shown in Fig. 1. It will be noted that the pumpage on the maximum day ranged from a little more than 4 mgd. to a peak rate of almost 15 mgd., or 375 per cent of the mini-

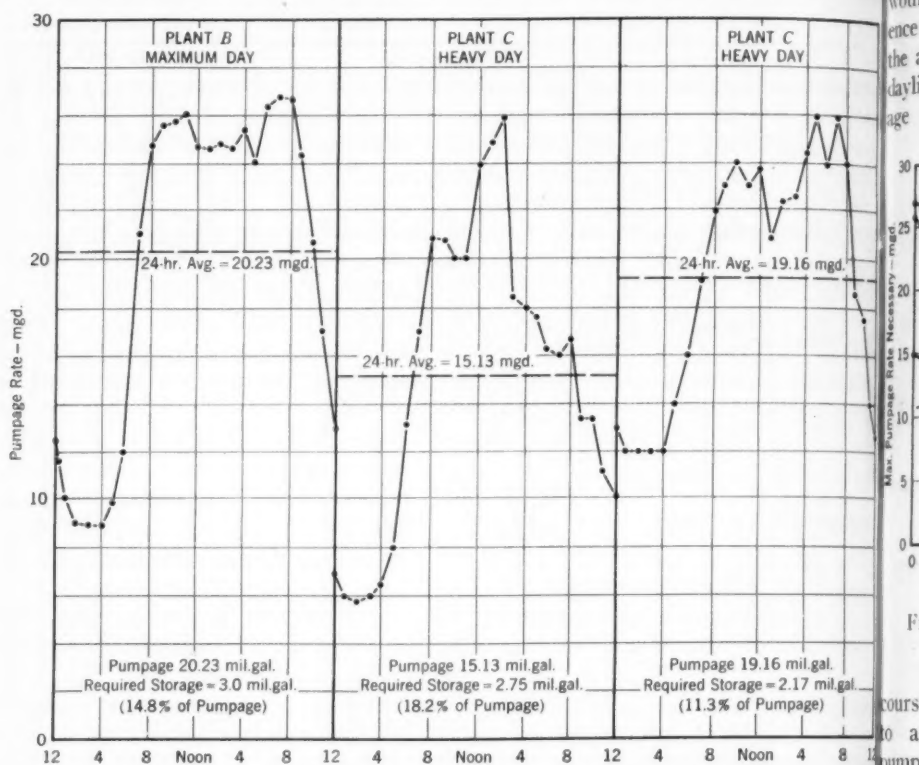


FIG. 2. Pumpage Variation and Required Storage—Plants B and C

be determined from a pumpage record extending over at least several years, because fluctuation from year to year is usually considerable. If the community is growing rapidly, it may be desirable to make a forecast of maximum day pumpage rates or at least to take into account the probability that rates will increase.

These conditions would require 1.9 mil. gal. of storage, or 17.6 per cent of the 24-hour pumpage, to supply the peak rate above the 24-hour average. By providing this amount of storage, it would be possible to operate the pumps at a uniform rate, which would come to 10.8 mgd.

To illustrate the saving involved, if a demand charge equivalent to \$1.25 per hp. per month is assumed, this reduction in the maximum pumping rate might result in a savings in demand charge of about \$2,500 per year.

The difference between the 24-hour average pumpage of 10.8 mgd. and the actual rate during the night hours would go into storage, and the difference between the 24-hour average and the actual peak hour rates during the daylight hours would return from storage to the distribution system. Of

to iron out the peaks of the maximum day. This amounts to nearly 39 gal. of elevated storage per capita, compared with about 28 gal. per capita for plant A of Fig. 1. The change reflects the difference between a city of diversified industry and a suburban community with characteristically high maximum day pumpage.

Plant C of Fig. 2, for May 21, shows pumpage characteristics calling for about 2.75 mil.gal. elevated storage, or 18.2 per cent of the total pumpage. This requirement, it is of interest to

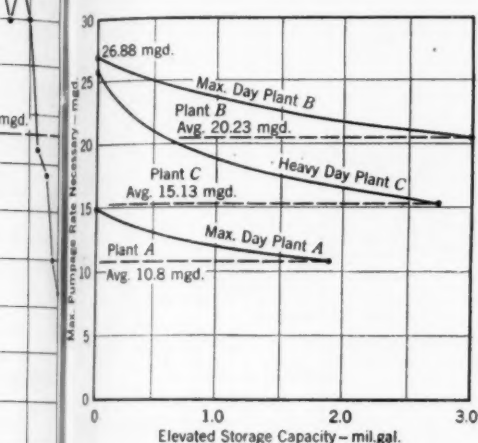


FIG. 3. Effect of Elevated Storage on Pumpage

course, it is not possible in practice to anticipate the maximum day's pumpage. Normally somewhat more elevated storage should be provided than the theoretical requirement, to minimize the effects of an error in judgment.

Figure 2 shows the maximum day's pumpage for plant B, supplying a suburban community of 78,000 population, and two typical days of heavy pumpage in plant C, supplying an industrial community of 160,000 population. Plant B would require 3.0 mil.gal. of storage

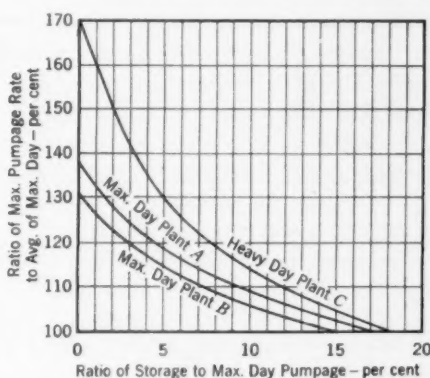


FIG. 4. Relation of Elevated Storage to Maximum Pumpage Rate

note, is for a day of less pumpage than that of Aug. 16, on which only 2.17 mil.gal. storage, or 11.3 per cent of the total pumpage, was required. Based on the figures for May 21, the required storage would be approximately only 17 gal. per capita, a low figure. It will be noted that the heavier pumpage on Aug. 16 was characterized by a high rate during the night, thus reducing the variation between night and day, so pronounced on May 21. This is a rather unusual situation, but shows that the maximum day in certain plants may not represent the governing condition in deter-

mining the amount of elevated storage necessary to smooth out the rate of pumpage.

Of course it is intended, in referring to the maximum day, to limit it to the maximum day of pumpage to meet the domestic demand, thus excluding pumping for fires.

The effect of elevated storage capacity in reducing the pumpage rate necessary to supply the demand on a given maximum day is illustrated in Fig. 3, which shows the benefit that may be obtained from any given amount of

mil.gal. of elevated storage and to 17.3 mgd. by providing 1.5 mil.gal. of storage. The maximum rate in Plant A could have been cut to 13 mgd. by providing 0.5 mil.gal. of storage and to 12 mgd. by providing 1.0 mil.gal.

For general use Fig. 4 is more convenient. This diagram shows the elevated storage in terms of the per cent of the maximum day pumpage, and the pumpage rates in per cent of the 24-hour average for the maximum day. It may be seen that the characteristics vary somewhat with different plants

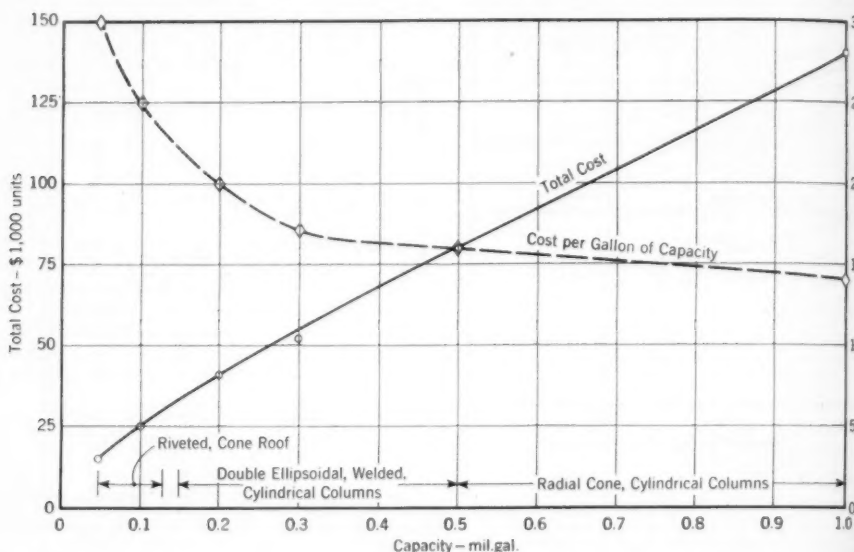


FIG. 5. Approximate Cost of Elevated Tanks With 100-ft. Towers

storage less than that necessary to iron out the fluctuations in maximum day pumpage completely. As applied to the maximum day of Plant B, it shows that the maximum pumpage rate could be reduced to 24 mgd. by providing 0.75 mil.gal. of elevated storage capacity, and to 22 mgd. by providing about 2.0 mil.gal. of storage. Similarly, in Plant C, the maximum pumpage rate could have been reduced to 20 mgd. by providing only 0.7

In Plant B, storage equivalent to per cent of the maximum day pumpage will reduce the peak pumpage rate required from 131 per cent to 114 per cent of the 24-hour average rate. Similarly, storage amounting to 10 per cent of the maximum day pumpage will reduce the rate from 131 per cent to 106 per cent; whereas storage amounting to 15 per cent of pumpage will permit practically uniform pumpage throughout the maximum day. When

cost is not a limiting consideration, elevated storage should of course be provided to the extent required to achieve this uniform pumpage, with a little added because in general it is not practicable to pump at exactly the 24-hour average rate of consumption.

If insufficient elevated storage is provided to iron out the peak rates of demand in the area served, it is sometimes desirable to valve off the tank for part of the day, in order to have the reserve storage available during the maximum peak hours, when it will do the most good.

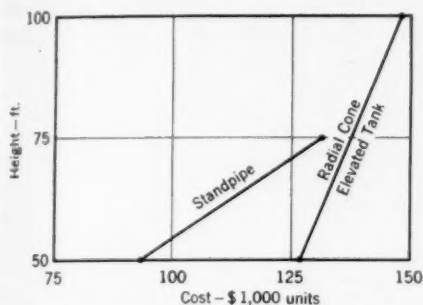


FIG. 6. Comparative Cost of Elevated Tanks and Standpipes

Elevated Tank Costs

Some idea of the cost of elevated tanks and of the economy of various sizes may be obtained from Fig. 5 which gives the cost, as estimated by E. E. Alt of the Chicago Bridge & Iron Co., of several sizes of tanks in terms both of cents per gallon and total cost, based on the construction of 100-ft. towers and tank complete, including foundations.

It will be seen that a 1-mil.gal. tank costs less than half as much per gallon as a 50,000-gal. tank, and that the reduction in cost per gallon is rapid when capacity is increased from 50,000 to 300,000 gal., the saving thereafter leveling off as size is further increased.

It appears, therefore, that tanks of less than 300,000 gal. capacity, when constructed in this manner, are relatively uneconomical. Such sizes, however, would be used only in the smaller communities. The relatively nominal saving effected in increasing capacity from 500,000 gal. to 1 mil.gal. shows that there is no great sacrifice in tank cost in providing several elevated tanks suitably located on the distribution system instead of concentrating the storage at a single point. Where storage of more than 500,000 gal. is involved, it is therefore proper to determine whether the total storage should be distributed among several tanks. This involves a study of the distribution system and, of course, the availability of suitable sites. The latter, rather than the distribution system, may well prove the determining factor.

In Plant B, serving a community of about 78,000 population, it was found advisable to provide two tanks, with capacities of 1.0 and 1.5 mil.gal. These were located nearly three miles apart, on the edge of the distribution system, each tank being about three miles from the pumping station.

Substitution of Standpipes

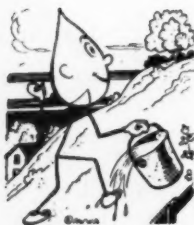
Where circumstances permit, the advantage of using a standpipe of about the same diameter instead of an elevated tank should not be overlooked. The standpipe may be justified where the site available is such that an elevated tank would be on a comparatively low tower. Up to a certain height, standpipes can be built for the same or less cost than 1-mil.gal. elevated tanks of the same diameter and height to overflow, as shown in Fig. 6. Although the storage in the standpipe below what would be the bottom of the elevated tank cannot be considered

useful to meet the fluctuations of demand during the maximum day, it does provide—practically without cost—a relatively large volume of additional storage which would float on the system during any emergency resulting in pressures below normal, such as a major break in the mains or a conflagration. Such storage should properly be credited with the cost of equivalent clear well storage, worth, say, \$45,000 per mil.gal. If this credit is taken into account, a strong case is made out for the low standpipe. At comparatively small expense, the reserve storage in the standpipe below the level of the alternative elevated tank bottom can be reclaimed by a pump to deliver it

to the distribution system during prolonged power interruption or other emergencies by providing a gasoline engine driven pump.

A 2.75-mil.gal. standpipe 65 ft. high was used in Plant A instead of a 1.75-mil.gal. elevated tank at about the same cost, thus providing approximately 1 mil.gal. of emergency reserve storage at virtually no added cost. The same plan, including the booster pump feature, has been followed in a number of cities.

The principal factors which limit the use of a large diameter standpipe to low height are excessive loading on the foundations and prohibitive plate thickness in the lower rings.



Progress in the Design of Elevated Steel Tanks

By Donald A. Leach

A paper presented on May 8, 1947, at the Indiana Section Meeting, Indianapolis, Ind., by Donald A. Leach, District Sales Mgr., Chicago Bridge & Iron Co., Chicago, Ill.

PRIOR to 1931, most elevated steel tanks were built in one of two general designs—employing either the ellipsoidal or hemispherical bottom, with cone roofs and structural columns. To improve their appearance, a few tanks had been completely surrounded with masonry towers, but, since the cost of providing the surrounding structure often exceeded the cost of the tank itself, the practice could not be considered economically feasible.

The late George T. Horton, then President of the Chicago Bridge & Iron Co., had over a number of years become convinced that, through attractive design and proper painting techniques, pleasing appearance could be achieved at a relatively small additional cost. To develop his idea, the company decided in 1931 to sponsor a competition for attractive elevated tank designs. A total of 152 designs, of which 8 were chosen as worthy of recognition, were submitted. In discussing the awards, Horton said:

Admitting that we as builders have failed to impart sufficient individuality to the various structures entrusted to us, we have recently sponsored the competition which developed the designs submitted. A few are manifestly impossible, while others are so economically unsound as to be impractical. A great majority of these designs can be utilized, many at

no great additional expense over standard designs.

The winner of the first prize made use of vertical pilasters to serve as columns, extending from the top of the tank to the ground, a feature which has since been used in many similar designs (Fig. 1).

Advent of Welding

Until 1931 the art of welding was very crude, and, because of the type of equipment in use, the scarcity of welders and the general lack of confidence in that method of joining metals, all elevated tanks were riveted. In 1933, the research department of the Chicago Bridge & Iron Co. made a complete analysis of the welding art and its potentialities for joining metals. Since the shielded-arc welding rod and equipment with controlled current were by then available, the tank industry decided to apply the method to field-erected flat-bottomed storage tanks. In practice it was found superior in many ways to riveting, allowing simple details and eliminating the leakage occasioned by improperly driven rivets.

Following the first successful application of welding, efforts were made to use the method on elevated tanks. Since most tank field forces were then composed of riveting crews, it was con-

sidered uneconomical to attempt welding of small tanks. There was developed, however, a radial cone type design, suitable for large capacity tanks (more than 500,000 gal. capacity), having a low range of head (25–35 ft.)

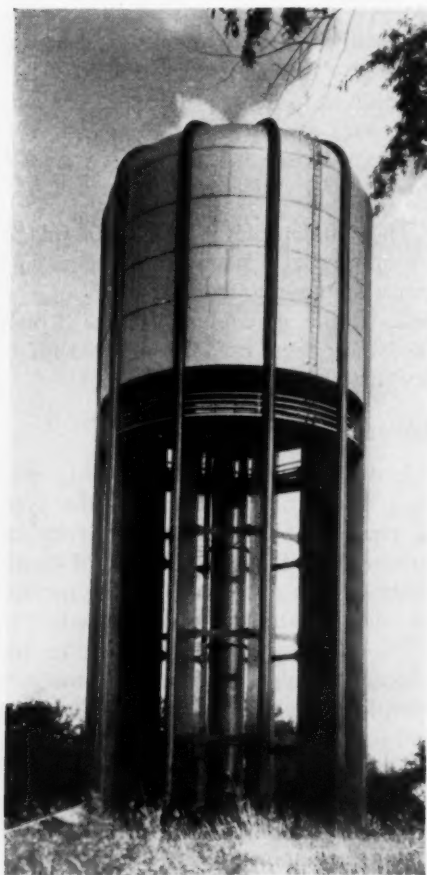


FIG. 1. Prize-Winning Design Utilizing Vertical Pilasters

and a large diameter. The term "radial cone" applies to the shape of the bottom, which consists of conical segments having their elements on radial lines sloping toward the center and supported on radial beams, all cone plates being in tension (Fig. 2). Whereas

riveted details for this type of tank were very expensive, welding adapted itself excellently to the connections between the bottom plates and radial girders.

Design Features

Tubular Columns

The first radial cone tanks were supported on structural columns. Later developments indicated the advantage of tubular columns, which were more popular because of their appearance and because they eliminate cross-bracing for heights up to 100 ft. Tubular columns are also hermetically sealed to eliminate the danger of internal corrosion, and



FIG. 2. Radial Cone Welded Tank

grit-blasted in the shop before painting to assure a paint coat of longer life; hence maintenance costs have been lowered.

Diagonal Wind Bracing

On any tower it is necessary to supply diagonal wind bracing, usually by means of solid round or square rods extending from the top diagonally to the bottom of the same bent. The design limit for the angle which the rod makes with the vertical is 30 deg.; this factor of course controls the design. It is necessary to plan the diagonal rods for tall towers with tubular columns and only one panel, so that they ex-

tend from the bottom of one post to the top of a second or third adjacent post. With multiple post designs, the diagonal rods thus installed give the appearance of the curve of an hourglass. They are straight elements in a surface of revolution and appear curved from any direction, thereby im-

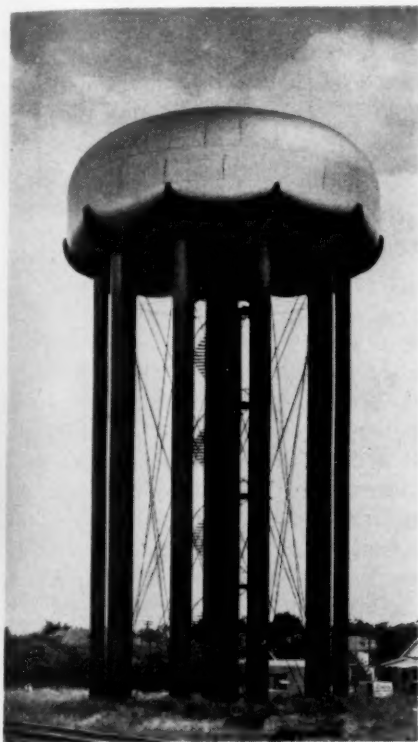


FIG. 3. Pattern Formed by Bracing Rods in Taller Tanks

proving the appearance of large tanks (Fig. 3).

Spheroid Structures

The newest development in the design of large capacity tanks is the elevated spheroid. It is made of plates of variable curvature in the shape of a spheroid, with no vertical shell. All plates are in tension and the tank must

have two sets of supports. The outer support is a circular girder supported on tubular columns. The inner support may be either a circular girder supported on additional tubular columns or a standpipe taking approximately one-third of the total load. This center cylinder must be stiff to resist the stresses; the use of fluted sections makes it capable of withstanding a much heavier load without additional reinforcement. This combination of the fluted center column with the outer tubular columns presents a very pleasing appearance. The tank shown in Fig. 4 is one which was recently completed for the city of Washington, D.C.; it is best suited to comparatively short towers.

Additional Designs

The strength of a tubular column is great enough to support tanks of small capacity without the use of additional outer columns. In the hope that such a tank could be built more economically than one with the outer columns, the watersphere was developed. This is a spherical tank with a cone frustum connecting the sphere to the supporting cylinder. The base of the cylinder is supported by an additional cone frustum which spreads the load to the foundations and gives greater resistance to wind-overturning moment (Fig. 5). Waterspheres are built in capacities up to 200,000 gal. on any height tower up to about 125 ft. The cost is greater than that of a tank supported by columns, but eventually it is hoped to develop a means of erection which will overcome this handicap. The present method of erecting the sphere is to assemble it complete about the riser, weld it in one piece, and then raise it to the top of the riser, using the riser as the support for the derrick. Originally the

large riser supporting the watersphere was filled with water, and the main connections were made at the bottom, as is done with post-supported tanks. In extreme northern localities, however, there is a possibility that the riser column may freeze solidly. In such climates, the tanks are now designed with a steel head plate in the top of the riser, fitted with an expansion joint and a small insulated cast-iron pipe extending from this head plate to the base of the riser.



FIG. 4. Tubular Columns With Fluted Standpipe

The reaction of different tank users is surprising. Many prefer the watersphere; others dislike and will not consider it. It may perhaps be instructive to recall a remark made by Horton some years ago, when the author took exception to one of his designs. He simply said "Perhaps I don't like your hat." Some designs appeal; others do not; but the constant demand for improvement has prompted all of them.

An interesting deviation from the watersphere design is that of the so-called "milk stool." This tank has the supporting tubular columns meeting at

the base of a cone bottom. The base is spread to give greater stability and no diagonal rods are necessary. Only two such tanks have been built, but the owners are quite pleased with their appearance. The design is shown in Fig. 6 to indicate what can be done with this combination of welded construction and tubular columns.

Additional designs prepared consist of large capacity elevated spheres supported by tubular columns, radial cone tanks with sloping columns and others which have never been built, but are possible, and at least have an unusual appearance (Fig. 7).

Small Welded Tanks

The large capacity tanks proved so popular with municipalities and engineers that it was decided in 1946 to apply the same principles of design to smaller tanks. It was also found that the supply of good riveting crews was limited and there seemed to be an abundance of good welders, developed during the war. Furthermore, certain jurisdictional disputes were encountered with the unions which could be eliminated by making a tank entirely of steel plates.

The outcome of these conditions is the double ellipsoidal welded tank, with tubular columns, in capacities down to and including 50,000 gal. All these tanks have a vertical shell, an ellipsoidal roof and bottom and as many panels as the height of the tower requires. The tubular columns all have a batter of $\frac{1}{2}$ in. in 12 in., which is less than that used for structural towers. This slope was chosen for appearance. The length of the column is again determined by the 30-deg. minimum angle which the diagonal rod makes with the vertical. The length of column can be determined to suit the design, because

the slenderness ratio is large in comparison with a structural column (Fig. 8). These tanks vary in capacity from 50,000 to 500,000 gal., with posts varying in number from four to eight. The provision of balconies is optional. The struts were originally made of tubular sections, but, because the erectors insisted on using them for runways, they were later changed to U-type box sections with flat tops for safety. The ap-

only gives sufficient vertical shear resistance to the load which it carries, but extends under the bottom as a cradle type support. The struts frame into the tubular column with gusset plates which are butt-welded to each member. The shoe consists of a flat slab of steel with two anchor bolt lugs welded to opposite sides. There is no pocket in the shoe at the base of the column in which dirt may collect. The use of

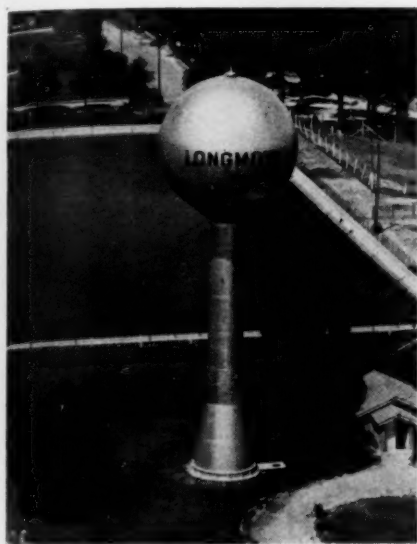


FIG. 5. Spherical Watersphere

pearance from the ground is that of a tubular section. For wash-water purposes, the welded tank with vertical tubular columns can be built with one panel in any capacity desired.

Simplicity of Details

Details which are used with welded tank construction are simple and easy to make. The post connection is made in the form of a cradle with half the tube cut away and mounted on a pad plate. This pad plate is, in turn, welded to the shell in the field. The pad plate not



FIG. 6. "Milk Stool" Variant of Watersphere Design

welding makes these details simple to construct.

All tubular columns are made in the fabricator's shop to insure proper alignment and proper welding. All plates are joined together with butt joints, except the joint between the roof plates, which may be lap-welded. Using butt joints below the water line eliminates any sharp corners which might invite corrosion. The weld metal is denser and more ductile than the parent metal; hence it is stronger and less

liable to corrosion than the body of the plate. Rivet steel, on the other hand, is softer than the parent metal and, if neglected, usually rusts faster than the parent metal. The maintenance cost of welded elevated tanks should therefore be lower than that of riveted tanks. The popular opinion is that welded

the oil industry. It is called a self-chalking tank white and is manufactured by many of the leading paint producers. This paint gives a welded steel structure a very clean appearance, causing it to blend with surrounding buildings and to appear less "commercial."



FIG. 7. Possible Design Employing Tubular Columns

tanks with tubular columns have a neater appearance than the riveted tanks with structural columns and look less commercial for residential areas.

Painting of Tanks

A new type of paint was developed several years ago, primarily for use by

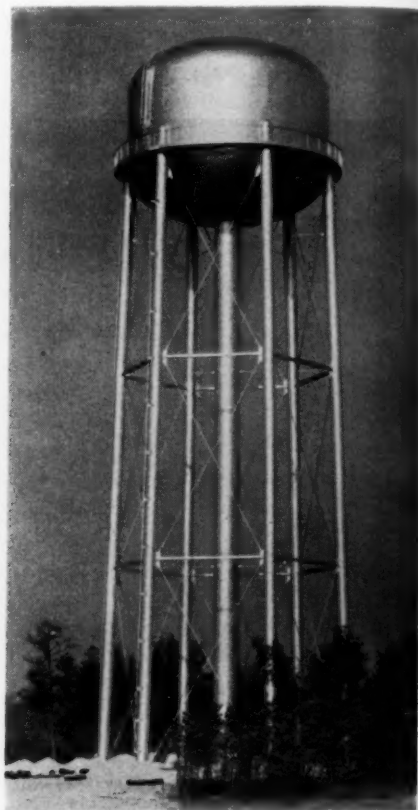


FIG. 8. Welded Tank of Small Capacity

A few months after application it begins to chalk, and cleans itself as the rains wash off the chalked pigments. The life of the paint is at least 3 years, and frequently it has lasted as long as 7 years without replacement or exposure of the under coat. Aluminum paint is still used and is much better

self-manufactured product for steel buildings. It is in appearance than the old standard dark green or black graphites. It is the general opinion that the new tank white makes a structure more pleasing in appearance than anything that has yet been developed.

This self-chalking white paint was developed, primarily, to reduce temperatures in oil tanks by reflection of the sun's rays. It was found that over a period of time the temperatures are lower in a tank painted white than with any other combination. Aluminum has a better record for the first few months

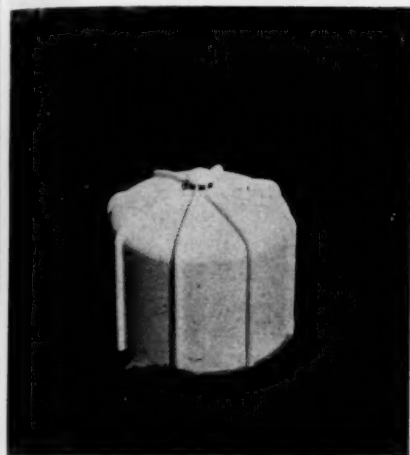


FIG. 9. Standpipe Made Attractive by Pilasters and Self-Chalking Paint

but the temperature rises as soon as it starts to turn gray; whereas the white paint is self-cleaning, keeping the contents of the tank cooler. For a tank containing drinking water, this feature is worthy of consideration.

Cathodic Protection

Frequently requests are received for advice about the use of cathodic protection in both new and old tanks. The author's personal experience is limited,

but he may observe that his firm thinks enough of the theory of cathodic protection to have installed it in its own tanks. It would certainly seem that the protection afforded is worth while, because nearly all specifications on which the company is required to bid for new elevated tanks include a cathodic protection unit as a part of the installation.

In order for cathodic protection to operate properly, the anodes must be immersed in the water in the tank, and only that portion of the tank in contact with the water is given protection by the cathodic unit. The manufacturers of cathodic protection equipment furnish a small unit for tanks that are completely painted on the inside and a larger unit for tanks that are not painted below the water line. It is believed that the proper protection is, first of all, good paint—kept in good condition—and, in addition, at the discretion of the owner, a cathodic protection unit. Manufacturers of this type of equipment can furnish the information required to determine whether cathodic protection is or is not justifiable for corrosion prevention under any specific conditions.

Standpipes

Although this discussion is concerned chiefly with elevated tanks, two developments which improve the design and appearance of standpipes may also be mentioned. Both make use of welded construction. For small installations, the addition of the ellipsoidal roof is all that is necessary to give a pleasing appearance, especially when the tank can be fairly well camouflaged with shrubbery and trees. For larger standpipes, it is necessary to break up the surface, which otherwise would have

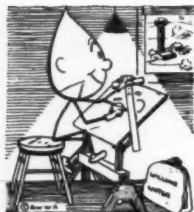
the appearance of a large billboard. This is done by means of vertical pilasters.

A standpipe of this design was built in Tulsa, Okla., in 1938. It is 85 ft. in diameter, 60 ft. high to the high water line, and has 8 vertical tubes, 3 ft. wide and 3 ft. deep, extending from the bottom of the standpipe to the curb; thence to the cupola vent house at the center. The cupola takes up the compression of the inner reaction of the tubes and the tubes break up the surface, improve the appearance, furnish enclosed overflow, access ladders and, in addition, support the roof without any internal framing. The tank is

painted with self-chalking white paint (Fig. 9).

Acknowledgment

Most of the designs illustrated are those of the Chicago Bridge & Iron Co.; the general types, however, are in use throughout the industry. It is hoped that progress may continue in the design of elevated tanks so that each structure built may be one that combines a pleasing appearance, comparatively low first cost and qualities which will require a minimum of maintenance. The attainment of such an objective will benefit both users and builders of elevated storage.



A Review of Illinois Water Development

By Louis R. Howson

A paper presented on April 17, 1947, at the Illinois Section Meeting, Chicago, Ill., by Louis R. Howson, Partner, Alword, Burdick & Howson, Chicago, Ill.

AT present 700 communities in the state of Illinois have public water supplies. They serve approximately 6,500,000 people through facilities costing nearly half a billion dollars. Most of these supplies have been built since 1880; the earliest was begun in 1840. A brief review of developments and trends during past years may help provide a better understanding of the future.

Early History

Operation of the first water works in Illinois was begun in Chicago by the Chicago Hydraulic Co. in 1840. The city of Chicago began the construction of its present water works system in 1851, but as late as 1897 there were two areas within the city—Pullman and Rogers Park—in which private water works still operated. The president of the Rogers Park company was H. E. Keeler, for many years secretary of the Illinois Water Supply Association, the predecessor of the Illinois Section of the A.W.W.A.

The public water supply at Chicago has always been taken from Lake Michigan, although until the late 1890's a great number of wells which had been drilled by industries continued to flow at the ground surface. By 1900 most wells in the Chicago area had ceased to flow and, in most of the more heavily pumped areas, such as the stockyards, the water level had receded to

as much as 100 ft. below the ground. Wells further removed from areas of heavy withdrawal continued to flow freely, however. Charles B. Burdick reported that (1) "As late as 1899 a 12-in. well at Dubuque gave a free flow of 1 mgd." Although the Chicago supply came principally from Lake Michigan, as late as 1900 it also included artesian well supplies for the annexed communities of Washington Heights and Norwood Park.

The second important water works built in the state was that at Springfield, in 1866, where the water works took its supply directly from the Sangamon River. This source was superseded, beginning in 1888, by an extensive system of infiltration galleries built along the river at about 25 ft. depth, which were in turn supplanted about 1910 by groups of shallow wells. Some twenty years later the city developed its present impounded supply.

The only other important supply in the state built prior to 1870 was that at Peoria, where in 1866 the city constructed a plant taking water from the Illinois River. Peoria at that time had a population of nearly 25,000. The selection of the Illinois River was based upon a report made by Octave Chanute, then Chief Engineer of the T. P. & W. Railroad, and later one of the best known engineers in this country. Chanute reported upon three sources of supply, which he designated as:

1. Copious springs on the opposite side of the river from Peoria.

2. Springs at the base of the bluffs on the city side of the river.

3. The Illinois River.

Some excerpts from his report have more than passing historical interest. Even then, more than 80 years ago, Chanute had analyses made of the various waters, reporting, for example, that the spring water contained:

... 28 grains to the gallon, an extensive quantity for a town supply. The tests show this to be mostly carbonates of lime, gypsum and a trace of magnesia, and an appreciable quantity of carbonic acid. . . .

The inquiry also occurs whether spring water is the best for the general purposes of a town supply. It is probably generally preferred for drinking because of its clearness and freedom from matters in mechanical suspension, and its equable temperature, being both cooler in summer and warmer in winter than the atmosphere. It was believed some years ago that the more nearly the water supply approximated to the condition of distilled water, the more healthy it was, but the theory of the German chemists now is that a small quantity of lime, say from 5 to 10 grains per gallon, renders water best adapted to human uses, and they instance the development and strength of all races inhabiting limestone countries as a fact in support of their theory. Not only is the water of all the Peoria Springs notoriously hard and very ill adapted for washing, causing a great cost in labor and soap, but it is also the general impression that it is not so well adapted for cooking as a softer water, being decidedly injurious to vegetables.

One very serious objection to the supply of a town from springs within its own limits remains to be mentioned. It is sure to be corrupted in time.

That Chanute was an economist as well as a scientist is evidenced by this statement: "In determining upon the

character of the works and estimating upon any plan, regard should be had rather to what is possible to effect than to what it is desirable to accomplish."

The Peoria supply from the Illinois River was financed through the sale of bonds which carried interest rates of from 7 to 10 per cent. Cast-iron pipe cost \$70 per ton; 12-in. pipe in place, with labor at about \$1 per day, cost \$3 and 6-in. pipe \$2 per ft.—both reasonably comparable with 1947 prices.

The city of Peoria continued to operate the plant until 1889, by which time the population had increased to 40,000, but, due to the poor quality of the water, there were still only 2,000 customers. The public's failure to accept the river water may not have been due to the Chicago practice of pumping part of its sewage to the Illinois River through the Illinois-Michigan canal, but it is known that, in November 1889, the year in which the Illinois State Legislature authorized the creation of the Chicago Sanitary District and the construction of the sanitary canal, the city of Peoria sold the water works to eastern capitalists who agreed to develop a ground water supply. The Peoria Water Works Co. has supplied the city from underground sources for more than 50 years, first from its wells at the main plant, and later extending its ground water development both north and south of the original station.

A number of other larger cities of the state started their water works systems a few years after the Civil War: Decatur in 1870 from filters on the Sangamon River, Quincy from the Mississippi in 1873, Rockford from deep wells in 1874 and Rock Island from the Mississippi in 1870. Of the now important Illinois cities which constructed plants prior to 1880, it is interesting to note that only Rockford drew its supply from underground sources.

Increased Construction

Beginning about 1880 there was a great expansion in water works construction. Baker's *Manual of American Water Works* for 1897 (2) lists just over 200 public supplies in Illinois, of which 19 per cent were surface supplies, including those of Chicago and several suburbs along Lake Michigan, and 81 per cent were from underground sources. In the development of these supplies several engineers, including D. W. Mead, John W. Alvord and Dabney Maury, all active in the A.W.W.A., played an important part. George C. Morgan, engineer, contractor and owner, built many of the smaller water systems which can be identified now by a stone or brick base tower, usually circular, which supports the steel tank.

By the close of the century there were 269 communities in Illinois with public water supplies. Of these, 77 per cent used ground water sources.

The *McGraw Water Works Directory* of 1914 lists 288 Illinois water works, although it is known that this listing is incomplete.

In 1938 the Illinois State Department of Public Health listed 625 public water supplies, of which 81 per cent used ground water.

As of 1947 there are approximately 700 water works in the state, of which 67 per cent are supplied from underground sources—almost evenly divided between drift and rock wells—and the remaining third from surface sources.

A study of the cities having populations of 10,000 or more in 1940 indicates that, at the present time, approximately 550,000 people in 19 cities are still using underground water, while the other 39 cities, with a population of approximately 4,000,000, use surface supplies.

Comparing the sources of supply in these 58 cities with their sources in 1910, it is found that, in 1910, 32 of the cities, comprising 17 per cent of the population of the group, depended on underground supplies. In the 30 years following, 15 communities, originally using underground water and having in 1940 a population of approximately 450,000 people, had changed from underground to surface supplies.

Expressed in another way, in the 30-year period from 1910 to 1940, the number of people using wells increased 5 per cent while the number using surface water increased 75 per cent. During the same period, nearly as many people had switched from underground to surface supplies as were still depending upon underground sources in 1940.

Selection of Supply

It is obvious from the simple record of procedure that in those cities which started their systems prior to 1870, the primary consideration governing the selection of supplies was visible adequacy. Pollution was then not too important a factor; buildings were largely of frame construction; fire protection was essential; and private wells were largely used for the drinking supply.

In the next two decades money was scarce, costs of construction materials were high, and the source of supply was often determined by selecting the cheapest expedient. Many cities used two or more sources. Galesburg had a supply consisting of 76 shallow tubular wells and 2 deep wells; Joliet had a combination of 26 driven wells and 4 artesian wells as late as about 1900.

The selection of the best source of supply for any city having more than one source from which to select is largely governed by economics. In the northern half of Illinois, few cities are

limited to a consideration of one source of supply. Deep wells are possible over this entire area; in many sections gravel wells are available for development, and, in most of the others, surface water is available either from stream flow or impounded development. The selection of the best procedure consists in first analyzing the needs of the community—past, present and future—outlining a plan of development of each practicable source to meet future needs, estimating the cost of installation now and for the near and distant future, and estimating the annual cost of water from each source, including in that estimate interest and depreciation on the investment as well as the direct operating costs, such as labor, fuel, power, and similar expenses. After the dollar comparison of the various practicable alternatives has been made, they must further be evaluated on the basis of quality. All supplies must, of course, be safe bacterially. If one supply is materially harder than another, they should be evaluated by adding to the construction and annual costs of the harder supply the expenditures necessary to yield waters comparable in quality from the chemical standpoint. Other considerations, such as temperature and its seasonal variations, physical appearance, freedom from tastes and odors and the cost of the necessary treatment, should all be considered before reaching a final conclusion. Finally, that means of supply is best which will most economically deliver the desired quality of water to the community, both present and future needs being considered.

It must be pointed out that the history of many large ground water supplies reveals a perennial shortage, resulting from a failure to obtain, in advance of requirements, a comprehensive

knowledge of underground conditions bearing upon the water supply. It is almost axiomatic that any municipality which does not provide for its water supply requirements in advance of its needs is habitually unable to meet those demands as they arise. In municipal water supply, ten years ahead may well be considered as the present. This failure to develop a comprehensive knowledge of the underground resources has led many cities to abandon ground water supplies and adopt visible surface water supplies, a decision not always wisely made. Failure to provide adequately for the requirements, while more common with ground water sources, is not, however, limited to them. There have been many instances in Illinois where impounded supplies failed either through inadequate consideration of the yield of the drainage area, insufficient spillway capacity resulting in dam failures or too-long deferred additions to storage capacity.

Another popular misconception of underground supplies is that they yield a naturally purified water, in contrast to surface supplies, which require treatment by man to make them safe. This misconception and the false feeling of security it affords against the hazards inherent in leaky well casings and other improper construction have—together with the dislike for chlorination—resulted in needless epidemics in cities using ground water supplies.

A ground water supply has one important advantage over an impounded supply in that *if properly planned* it can usually be extended progressively in pace with and ahead of the increased requirements. Unfortunately it is all too often not properly planned and built in advance of demonstrated need.

Impounded supplies may be constructed either on large watersheds

where the silting of reservoirs is an important but frequently overlooked consideration, or on smaller drainage areas, sometimes adjacent to a major stream, where deficiencies in yield of the small area can be supplemented by pumping to the reservoir from the main stream, thus avoiding the silting problem.

Surface supplies experience taste and odor difficulties not generally found in underground waters.

Behavior of Ground Supplies

In recent years much has been written in the daily press which conveyed the impression that there was a general failure of underground water supplies in Illinois. That impression is far from correct. Illinois is not an arid state.

The impression arises primarily from the difficulty of withdrawing increasingly large quantities of ground water in a few isolated areas. These difficulties generally arise from increased demand which evidences itself in different ways, depending upon the source of the underground supply.

Localities obtaining their supplies from deep rock wells usually experience a rapid drop in water levels with a sudden increase in demand. This results from the increased frictional resistance in the pores of the rock, creating a steeper gradient near the well. Such difficulty was experienced at the Kankakee Ordnance Works, where a considerable number of deep wells were drilled early in the war. When pumped at a rate of 9 mgd. the water level dropped more than 100 ft. in 30 days. When the pumping rate was reduced, the water level rose. The obvious solution of the difficulty was to limit the pumpage from the wells to a rate which would maintain the running

water level within reach of the pumping equipment and secure all additional water from the nearby Kankakee River.

In shallow gravel wells, the water ordinarily enters the ground in the immediate vicinity of the strata. The gravels with their voids serve as storage reservoirs and as transmission media for the water from the point where it enters the strata to the point where it is withdrawn. The quantity of water that can be withdrawn continuously in a given location depends upon numerous factors, among the most important of which are:

1. Rainfall in the locality of the collecting area. In Illinois this averages about 35 in. per year, of which about 25 per cent runs off into the streams and the other 75 per cent percolates into the ground.

2. The extent of the collecting area tributary to the water-bearing strata.

3. The transmission capacity of the strata themselves. The coarser, cleaner and more uniform the materials, the more readily they transmit water.

When the draft exceeds the replenishment rate, the water level in gravel wells is depressed much as it is in the rock wells, although usually much more gradually, because of the greater porosity and storage in the gravel. In most of the more acute locations, the difficulty results from the fact that the natural water resource is being "mined" rather than "developed." Under such conditions, gravel wells may be completely exhausted.

The Peoria area is an example of such depletion of water resources. In this region, the public water supply draws approximately 13 mgd. from the gravels; and approximately three times as much is pumped by industries and used largely for cooling purposes. The

amount withdrawn exceeds the ability of the area to recharge, and, as a result, the water level in the industrial area immediately adjoining the Illinois River is frequently drawn down to as much as 30 ft. below the river. This situation could be corrected by reducing the draft or by using artificial recharge under properly controlled conditions. There are comparatively few locations in the United States where as much as 50 mgd. can be continuously drawn from the gravels or sands. At other locations, such as Houston, Tex.; Long Island, N.Y.; Baton Rouge, La.; and Louisville, Ky., it has usually been found necessary to seek surface supplies when the demand becomes so great.

The Champaign-Urbana area offers another illustration of overdraft. Well water has been drawn from the underlying gravels in this area by the public water supply, the University of Illinois and industries for a period of 61 years. During that time, over an area of approximately 100 square miles, the water level in the gravels has been lowered a maximum of approximately 90 ft. and to an average depth over that area of approximately 15 ft. That the immediate area has been overdeveloped is apparent. Explorations not yet completed indicate that the extent of the gravel beds is much greater than the present development. The water company is proceeding with the construction of additional wells and pipelines in order to spread the development and keep the withdrawal within the replenishment rate for the area.

Future Supplies

In making any prediction concerning the future sources of water in Illinois, it must be remembered that in the southern third of the state there is

in general no choice, for ground water is available only in isolated locations. Surface supplies, largely impounded, will predominate.

In the northern third of the state, there are usually three possibilities: deep wells, shallow wells and surface supplies. Those cities within reach of Lake Michigan will undoubtedly utilize that source of supply. The outer rim of the area to be supplied from Lake Michigan is dictated by economics and today is believed to extend not much further west than the Cook County line. In the author's opinion, it is doubtful whether the Fox River cities will be supplied from Lake Michigan in his time.

As the mineral quality of water supplies is more generally appreciated and expressed in dollar terms, greater consideration will be given to softening. When cities using underground sources are faced with major expansion programs, they should have comparative studies made of further expansion of the underground supplies with softening and the cost of developing surface supplies with filtration and softening where desirable. If softening is indicated, filtration of surface supplies may sometimes be preferable to further development of well supplies. In many sections of Illinois, the hardness of the surface and shallow underground supplies is substantially the same. Softening can be added to filtration with but a nominal increase in construction cost, usually less than 15 per cent, and the chemical cost of softening the well water is usually comparable to that of the surface water. Consideration of softening may well be a determining factor in a decision between continued ground water development and the development of

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a new surface supply in some localities in the northern third of the state. It is in the central division that there has been a more pronounced transition from ground water to surface water supplies; Springfield abandoned both its wells and original softening plant when it developed its extensive impounded supply for both power plant condensing water and water supply; the surface development included a new softening plant. Bloomington built an impounded supply to supersede an inadequate shallow ground water supply of extreme hardness. In the Champaign-Urbana area an extensive study, which included comparison of impounded supplies at three locations with underground development from areas outside the influence of the present wells, has led to the exploration of underground resources as much as 15 miles distant from the present development, and the immediate construction of three large gravel-packed wells at an intermediate location 6 miles distant from the present development. It is believed that the Champaign-Urbana requirements, including the university and all industries, can best and most economically be supplied to as late as 1980 by a con-

tinued expansion of the ground water development.

Conclusion

The selection of the best source of water supply in localities where more than one source is available is and always will be largely an economic matter. Such a determination can be much more adequately made in 1947 than it could have been made when most Illinois water works were first built. The data now being compiled by such agencies as the Illinois State Water Survey, the Illinois and U.S. Geological Surveys and the State Dept. of Health, and the growing availability of maps and data on water resources, both above and below the surface, make it possible to proceed with greater assurance than ever before. There are ample water resources in Illinois. The problem is how best to select and develop them in the public interest.

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Equitable Schedules of Water Rates

By Albert P. Learned

A paper presented on Mar. 13, 1947, at the Kansas Section Meeting, Wichita, Kan., by Albert P. Learned, Cons. Engr., Black & Veatch, Cons. Engrs., Kansas City, Mo.

WATER rates are a means for billing the cost of preparing and delivering water to customers. Expedience and justice require that such rates distribute these costs equitably, but rates have not always been carefully developed, or revised to meet changing conditions.

The operating organization of a water works system is seldom concerned about the rates charged unless a severe complaint is registered by the customers. There are several reasons for this apathy. Water is essential to life. No substitutes exist, as they do, to a certain degree, for such utilities as electric and gas services. Low rates have been charged for water because the commodity is essential, and the customer has used what he needed.

Except for air-conditioning equipment, no domestic appliance has been developed that has materially increased water consumption, or offered added convenience in its use. Possibly the increase in family incomes in recent years may have permitted the maintenance of more attractive yards in the summer, thus requiring more water. Such increased usage has placed added requirements on the supply system and has not tended to level out the annual load factor, as has been true of the electric industry, in which the use of new appliances has tended to equalize

day and night and other daily or seasonal variations in consumption.

The savings in the cost of pumping, due to the increased use of electric power and the reduction in its cost through the years, has been offset largely by the increases occasioned in meeting public health requirements for better water and also, at times, the demand for softer water.

Growth within cities and revised usage of older areas has often required an enlargement in the distribution facilities to meet the requirements of general service and fire flow. Fire flow requirements have increased as the population of cities increased, for fire hazards are greater and more dangerous in congested areas, and it becomes necessary to protect ever larger and more valuable structures.

The present upward surge in both material and labor costs has forced the subject of rates to the attention of water works men. The out-of-pocket expenses for operation and maintenance have taken a larger part of gross receipts, and the revenue remaining for extension has bought materially less per dollar invested. The lowered rate of interest on borrowed funds has offset somewhat the upward surge in material and labor costs, but this has been of no help to a utility accustomed to finance its entire operations, includ-

ing construction of extensions, out of current income.

Two particular rate practices merit discussion before the methods of equitable distribution of costs are analyzed in detail. These are the policies of reducing rates during the summer season, and of allotting only a small allowance—if any—for fire protection costs to the water department.

Lower Summer Rates

The lowered summer rate for water is intended to subsidize the maintenance of more beautiful yards and grounds. It has merit from the standpoint of community beauty; it has little merit from the standpoint of equitable

may be cited. It is indicative of what lowered rates can do in an arid country. One of the rate schedules, in addition to the regular summer discount of 10 per cent, permits an additional discount of 15 per cent from May 15 to September 15 if the customer maintains a lawn on the parking strip. The base schedule, used by that city during the winter, is as shown in Table 1. A previous rate, effective from March 15 to October 15 and available to residential customers maintaining a lawn in the parking strip, is shown in Table 2. This rate was also subject to a 10 per cent summer discount.

These rates involve a decided reduction for the residential consumer who uses considerable amounts of water for

TABLE 1
Winter Rate Schedule

	Quantity cu.ft. per month	Cost per 100 cu.ft.
First	1,000	\$0.25
Next	1,000	0.15
Next	48,000	0.10
Next	50,000	0.08
Next	100,000	0.06
Over	200,000	0.05

TABLE 2
Former Winter Rate Schedule

	Quantity cu.ft. per month	Cost per 100 cu.ft.
First	800	\$0.25
Next	1,200	0.10
Over	2,000	0.05

distribution of costs of service. Such rates are usually inaugurated in an arid or semi-arid area, and with few exceptions are in effect at a time of the year when the water supply is lowest. There are communities where this practice has resulted in a customer demand far in excess of the maximum fire flow requirements, thus necessitating a substantial enlargement in the supply works, treatment plant, transmission lines, reservoir capacity and feeder mains within the distribution system—all designed to meet the demand for a relatively short period of the year.

An example of this situation, in perhaps a somewhat aggravated form,

irrigation, and it is evident, from a study of the water consumption in this city, that the increased usage has largely been applied to the increased area irrigated.

The variation in consumption and demand on the system through the years can only be reconciled when climatic conditions are considered. The actual peak demand on this system is the maximum requirement for purposes other than irrigation plus an amount, during the irrigating season, equal to 22.3 in. of rain on the area actually irrigated. Where such climatic conditions exist, the application of these lower rates—at a time when the plant's capacity is already heavily

taxed—can make an enlargement of the supply facilities necessary. An increased investment is thus required for facilities that are needed for only a short period of the year, when the rate of income they earn is actually lower.

It would certainly seem that a more equitable arrangement would see that the rate applicable during this period should provide sufficient additional revenue to pay for the extra facilities. Such a rate, which would apply during the entire year, is suggested for the community cited, as shown in Table 3.

The only excuse for a special summer rate is its psychological effect. If water can be produced at the lower costs, a fairer and more sensible policy

TABLE 3

Proposed General Rate Schedule

	Quantity cu.ft. per month	Cost per 100 cu.ft.
First	400	\$0.25
Next	600	0.20
Next	1,000	0.15
Next	48,000	0.10
Next	50,000	0.08
Next	100,000	0.06
Over	200,000	0.05

would be to make the savings available throughout the year. Such application may result in a little more off-seasonal use, and so would tend to equalize seasonal demands. A proper rate for added use will avoid the discrimination usually shown in a special summer rate.

Fire Protection Charges

Another controversial issue is the handling of the fire protection charges. The two main subdivisions of service furnished by a water department are general service and fire protection service. It is true that the investment in special facilities, such as hydrants

and their connections, is not large, and that the quantity of water used for this service during the year is not a large part of the water produced. Nevertheless, the supply, pumping, storage and large feeder main facilities must be of sufficient size to meet the large fire flow requirements. Fire protection is largely a stand-by service, and its costs are primarily capacity costs.

The furnishing of fire protection service free by the municipal system can be considered as a service rendered in lieu of taxes. Such a method, however, does not result in equitable distribution of costs. Fire protection value to the community and the individual is measured primarily by the property one has that can be destroyed by fire. The absentee owner of a large building that is leased to tenants has a vital interest in fire protection, but he does not contribute a cent for that protection if it is provided free by the municipality, and the the only water customers in the building are the tenants.

Municipalities supplied by private companies have often objected to the amount of their hydrant rental charges, particularly when growth required the installation of additional hydrants on existing mains and the charges depended upon the number of hydrants connected to the system. In at least one community these additional hydrants are charged at a materially lowered annual rental. At the present time, hydrant rentals do not constitute as large a proportion of the total income in privately owned systems as they did many years ago. Some of this reduction was warranted, and some of it has been forced below the point of equitable distribution of costs by a desire of the public officials to keep taxes down.

Costs of Service

A method of measuring the fire protection service costs of a water department is to break down water costs into: (1) capacity or demand, (2) commodity and (3) customer costs, similar to the methods used in the study of electric costs.

Demand or capacity costs include the fixed charges and operating expenses incurred in building the facilities that meet peak requirements. Commodity costs include the costs that vary with the volume of water that is actually produced. Customer costs, which are not dependent on the quantity or rate of use, include fixed charges on that part of the property not intended for peak requirements and the expenses incurred in its operation, together with the cost of customer meter reading, billing, collecting and accounting.

The physical property that is designed to meet peak requirements, and so constitutes the base on which the fixed charges are determined for capacity or demand, includes the supply and treatment works, pumping equipment, reservoir, transmission lines and distribution lines above 6 in. in diameter. The physical property considered in customer costs include all distribution lines 6 in. or less in size, services and meters. Hydrants and hydrant connections are included in fire protection plant. General plant is usually allocated on a dollar ratio to the above subdivisions. The dividing line between capacity and distribution investment in mains is set at 6 in. because this size pipe is used in many locations to insure satisfactory service to the customer, and is not a size determined entirely by fire flow requirements.

The operating and maintenance expenses included in capacity costs are

those listed in the *Manual of Water Works Accounting* (1) as:

1. "Source of supply expenses"
2. All items of expense under "power and pumping expenses" except fuel and purchased power
3. All items in "purification expenses" except chemicals which are included in supplies
4. All items in "transmission and distribution expenses" that apply to transmission lines and to distribution lines over 6 in. in size.

The operating and maintenance expenses included in commodity costs are those designated in the *Manual* as:

1. Fuel and purchased power under "power and pumping expenses"
2. Chemicals included in "purification expenses."

The operating and maintenance expenses included in customer costs are:

1. All items under "transmission and distribution expenses" not included in capacity costs or related to hydrants, fire mains and other fire protection plant
2. "Customers accounting and collecting expenses."

The operating and maintenance expenses that are included in fire protection are:

"Maintenance of hydrants, fire mains and other fire protection plant in transmission and distribution expenses."

The administrative and general expenses are distributed among these categories in part on the basis of labor costs in the respective subdivisions, in part on the basis of customer costs, in part on the basis of the property involved, and in part on the basis of the total expense incurred prior to these expenses, excluding fuel and purchased power.

The fixed charges include a provision for depreciation and return which

is a function of the cost of the property involved. An appraisal may be required to determine the base on which depreciation and returns are to be calculated. Taxes follow the allocation of the property items.

The municipal utility uses this return either for the payment of outstanding debt, retirement of debt or as a means to raise money for improvements and extensions. In its annual statement, one well-operated municipal water department includes depreciation in its operating revenue deductions, and, under the heading of disposition of net income, apportions a reservation of income for extension and betterment. This reservation is approximately 1.25 per cent of the total cost of the plant. On the balance sheet, under liabilities, the unspent portions of these funds are carried as a reserve for extensions and betterment; at the present time this reserve is equal to about 7 per cent of the total plant.

Most water works systems require very little replacement, leaving the depreciation reserve also available for extensions. In the plant mentioned, the total amount of the annual depreciation plus the annual reserve for extensions and betterments is approximately 2.5 per cent of the total cost of the utility plant. The debt service of this utility is very low, but in some utilities, less favorably situated financially, the money creating the reserve for extension would be absorbed for debt service.

Demand costs must be allocated according to the demand which the customers are capable of placing on the system. These demands are usually determined by the size of the meter connections. The demand of an unmetered connection for a sprinkler system in a privately owned building may

be determined by the size of the service line.

There are so many large hydrant connections in a city that, if each of these were included as a potential user, fire protection would bear an excessive proportion of capacity costs. It has been found that a reasonable measure of this potential demand can be determined if the number of fire hydrant connections included in the demand calculations be sufficient to produce the required fire flow. This places the fire demand on an equitable basis with general service requirements.

TABLE 4
Capacity Factors for Different Meters

Size of Meter <i>in.</i>	Safe Maximum Rate per Minute <i>gpm.</i>	Capacity Factor
$\frac{5}{8}$	20	1.0
$\frac{3}{4}$	34	1.7
1	53	2.7
$1\frac{1}{4}$	75	3.8
$1\frac{1}{2}$	100	5.0
2	160	8.0
3	315	15.8
4	500	25.0
6	1,000	50.0

It has proved to be equitable to allocate demand costs on the basis of the sum of the potential general service and fire protection demands. A very large percentage of all customers receive water through $\frac{5}{8}$ - or $\frac{3}{4}$ -in. meters, and it is not unfair if a diversity factor is applied to these residential customers, on the assumption that their potential total demand will not exceed 80 per cent of the sum of their potential individual demands. The diversity is due to the large number of customers in this group.

The capacity factors used for different meters—before the application of the diversity factor—to determine the

potential demand on the system by the customers, are listed in Table 4.

The use of this table allows the reduction of potential demand of all customers to $\frac{3}{4}$ -in. meter units, and the demand costs can be allocated to the respective customer in accordance with his demand. Fire protection is factored in by the amount of potential demand that the required fire flow creates, and can be calculated from the number of outlets required, as outlined above. One could reduce this demand to a gallonage basis by using the amount shown under safe maximum rates per minute. The use of a $\frac{3}{4}$ -in. meter discharge as unity, however, will simplify calculation.

Fire protection will comprise from 15 to 35 per cent of the total capacity charge. It will include little of the commodity cost and practically none of the allocated customer cost. Direct costs of hydrant maintenance will be a part of the total to be included in fire protection costs.

Commodity costs are allocated on a volume basis, charging an equal amount to each 1,000 gal. or 100 cu.ft. delivered.

Customer costs are ordinarily allocated to each customer in equal amounts, unless there is some special reason for them to be divided differently. The assumption is that it costs the same amount of money to read, calculate, post and collect a small bill as a large bill.

Rate Schedules

The allocation of costs to classes of customers gives the total costs of service for the particular classifications. After these have been made, rate schedules may be developed to produce the revenue.

Many hold that it is much fairer to make a readiness-to-serve charge and

to add to it the actual water consumed at the commodity cost. This means that the constant costs which are not influenced by the amount of water used would be borne equally, and a customer who used more water would pay for his extra usage at a low rate. This customer, however, would already have paid for his constant costs. There is considerable merit in the method, but it would result in a larger charge to the minimum user. Little criticism can be directed at such a charge from the cost angle. The situation is partially taken care of in the monthly minimum charge, based on the meter size, but this is seldom adequate, and part of the demand and customers costs are reflected in the lower-priced brackets of the rate schedule. It is proper, however, that the rate schedule be so designed that, as soon as a customer has taken care of his constant costs, he will be able to purchase at a price equal to or close to the commodity cost.

One city that makes an effort to secure a considerable part of its constant costs in the minimum charge uses the schedule of minimum charges shown in Table 5.

The minimum charge in this schedule is increased 20 per cent for customers outside the city. The price per 100 cu.ft. is not, however, increased correspondingly. The minimum rate is also reduced for 4 and 6 in. connections in some localities where the service is for connections to automatic sprinklers, private hydrants and standpipes used for fire protection. This charge is sometimes not over 40 per cent of the amounts in Table 5 if for the automatic sprinkler service. It is probably considered as an additional fire protection facility, which may reduce the drain on the system in case of fire.

The present upsurge in labor costs is not likely to be as serious to water department operating expenses as it would be in an electric system, because water works labor costs will ordinarily absorb a smaller portion of each dollar of revenue. The upsurge does have an important bearing on the cost of improvements to plant, and this increase in construction costs will exceed the average for the electric industry. Careful management may, however, be able to avoid an increase in water rates.

Sewerage Rates

The maintenance and operation of a sanitary sewer system and sewage dis-

TABLE 5

Schedule of Minimum Charges

Size of Meter in.	Monthly Minimum Charge
$\frac{5}{8}$	\$ 1.00
$\frac{3}{4}$	1.50
1	2.00
$1\frac{1}{2}$	5.00
2	10.00
3	15.00
4	20.00
6	30.00

posal plant require funds. Communities that have seen fit to handle these costs—as well as those of building such facilities—on a rate basis, have found that collection and accounting procedures are made much more simple and economical when both the water and sewerage service charges are handled by the same department.

The water carriage sewer system has made sanitary sewage consist primarily of the spent water supply of the community. Except for such water as is used for irrigation, lawn sprinkling, fire protection and some industrial

uses, the water “consumed” is returned to the sewer system. A monthly charge of 50¢ for each residential connection for sewer system operation, maintenance and amortization has proved sufficient in a number of communities where the sewer system included only the collection and primary treatment of the sewage. If extensive secondary treatment is required, this cost may be increased materially, and may almost be doubled. Commercial concerns that do not produce strong sewage can be charged approximately 10 per cent of their water bill, except if the water becomes a part of the product, as in an ice plant. For such plants, it may be necessary to increase the charge to 20 per cent of that portion of the water bill that represents water not going into the product.

Industrial wastes may materially affect the cost of disposal and may necessitate a special rate for allocating the proper cost to such customers. As a general rule, sewage for the entire community can be handled at a lower cost rate than sewage for an industrial concern can be handled individually, because very often the by-product produced by the industrial concern is so diluted when handled by the system as a whole that it creates no particular serious loading on the system. Furthermore, one large structure can treat sewage more cheaply than several smaller ones.

Sewerage charges should be analyzed just as carefully as water rates, to insure an equitable distribution of these costs.

Reference

1. *Manual of Water Works Accounting*. Municipal Finance Officers Assn., Chicago, & Am. Water Works Assn., New York (1938).

Kansas Plan for Neosho River Basin Development

By R. V. Smrha

A paper presented on March 13, 1947,¹ at the Kansas Section Meeting, Wichita, Kan., by R. V. Smrha, Sr. Engr., Div. of Water Resources, State Board of Agriculture, Topeka, Kan.

SINCE the turn of the century, several surveys and investigations have been made and plans prepared for works which would provide protection to land and improvements lying in the flood plain of the Neosho River. One of the earliest surveys of the flood problem on this stream was made in 1908 by the Office of Drainage Investigations, U.S. Dept. of Agriculture (1). In 1911, a similar survey of the Cottonwood River was made by the same agency at the request of the citizens in Marion. As a result of these surveys, flood control plans were proposed to provide for removal of obstructions from the river channel, construction of levees on either side of the channel with a minimum floodway of 900 ft. along the lower reaches and the cutting off of a few bends in the upper section of the river. None of these plans, however, was adopted; and, in the two decades which followed, the problem was further aggravated by the practice of building additional levees immediately adjacent to the stream.

Proposed Levees

Following the great floods on the Mississippi River in 1927, Congress authorized the Corps of Engineers to make surveys of all tributaries of the Mississippi River which were subject to destructive floods. The engineers' report, House Document No. 308,

74th Congress, first session, was transmitted to Congress by the Secretary of War on July 29, 1935. This report recognized the flood problem on the Neosho River, but concluded: "that while flood control has not been found to be economically justifiable at the present time, the most practical plan for flood control appears to be the construction of levees in the upper reaches above the mouth of the Spring River" The report also stated "that the flood problem is of local interest and no federal interest appears to be involved."

Prior to 1936 it had been the policy of the Congress to authorize the construction of levees along such streams as the Mississippi for the purpose of confining the flow for navigation. Congress did not recognize a federal interest in the control of floods where navigation was not involved.

In the Flood Control Act of June 22, 1936, the following policy was declared:

SEC. 1. It is hereby recognized that destructive floods upon the rivers of the United States, upsetting orderly processes and causing loss of life and property, including the erosion of lands and impairing and obstructing navigation, highways, railroads, and other channels of commerce between the states, constitute a menace to national welfare; that it is the sense of Congress that flood control on navigable waters or their tributaries

is a proper activity of the federal government in co-operation with states, their political subdivisions and localities thereof; that investigations and improvements of rivers and other waterways, including watersheds thereof, for flood control purposes are in the interest of the general welfare; that the federal government should improve or participate in the improvement of navigable waters or their tributaries, including watersheds thereof, for flood control purposes if the benefits to whomsoever they may accrue are in excess of the estimated costs, and if the lives and social security of people are otherwise adversely affected. . . .

SEC. 3. That hereafter no money appropriated under authority of this Act shall be expended on the construction of any project until states, political subdivisions thereof or other responsible local agencies have given assurance satisfactory to the Secretary of War that they will (a) provide without cost to the United States all lands, easements and rights of way necessary for the construction of the project, except as otherwise provided herein; (b) hold and save the United States free from damages due to the construction works; (c) maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of War. . . .

In line with this policy, Congress authorized the construction of certain flood control works. Included in this authorization were levees to protect the Kansas cities of Florence, Cottonwood Falls, Emporia, Neosho Rapids, Hartford, Burlington, LeRoy, Neosho Falls, Iola, Humboldt and Chetopa. Also included were levees to protect two agricultural areas: one in Cherokee County, east of Chetopa, consisting of 10,920 acres; and one in Lyon County, consisting of 7,020 acres near the town of Hartford. The plan called for federal funds to construct the levees if the benefited areas met the requirements

of Sec. 3, Flood Control Act of June 22, 1936.

Hearings were held by the District Engineer, Corps of Engineers, in several towns in the Neosho Valley in the summer of 1936, to ascertain the amount of local participation forthcoming. At these hearings it was determined that, with the exception of the city of Iola, local interests did not desire the construction of the proposed levees and would not provide the necessary rights of way.

Corps of Engineers Survey

As a result of the general dissatisfaction of the valley inhabitants with the recommendations of House Document No. 308 and the authorized flood control works, the Congress authorized and directed the Secretary of War to cause a preliminary examination and survey to be made for flood control on the Neosho River and its tributaries in Kansas, Oklahoma, Missouri and Arkansas.

The resulting report, dated January 15, 1945, prepared by the Tulsa District Engineer, recommended the construction, at federal expense, of four dual-purpose reservoirs in the Neosho Basin in Kansas. The selected locations are: (1) on the Cottonwood River above Marion, (2) on Cedar Creek above Cedar Point in the Cottonwood Basin, (3) on the Neosho River above Council Grove and (4) on the Neosho River below its junction with the Cottonwood and a short distance above the town of Strawn. The report was recommended for approval by the Division Engineer and the Board of Engineers for Rivers and Harbors, and was submitted by the Chief of Engineers to the Governor of the State of Kansas in accordance with the provisions of Public Law 5334,

78th Congress, second session. It was then referred to the special advisory committee provided for in House Concurrent Resolution No. 5 of the 1945 session of the Kansas Legislature. The Governor, after a study of the recommendations of the committee, approved the report and submitted his written views and recommendations to the Chief of Engineers.

Soil Conservation Service Study

In addition to the studies made by the Corps of Engineers, the U.S. Department of Agriculture, under Sec. 2, Public Law 738, 74th Congress, was given the following authorization:

SEC. 2. . . . federal investigations of watersheds and measures for runoff and waterflow retardation and soil erosion prevention on watersheds shall be under the jurisdiction of and shall be prosecuted by the Department of Agriculture under the direction of the Secretary of Agriculture, except as otherwise provided by Act of Congress. . . .

Surveys and investigations in the Neosho Basin were begun in 1941 by the Soil Conservation Service, under the provisions of this act. Work was continued until 1943, at which time it was discontinued because of the war. It is now being resumed and, when the report is completed, it will be submitted to Congress for consideration.

It is expected that the application of land use and vegetative measures will reduce the small but frequent floods on small tributaries but will produce only slight reductions in the large floods along the major streams. It will also cause some reduction in the sediment load of the streams.

A State Plan

The importance of water as a natural resource was recognized by the

state legislature three decades ago, when it created the Kansas Water Commission in 1917 for the purpose of bringing about the orderly development of the state's water resources. The preparation of plans for such development became one of the functions of the Division of Water Resources of the Kansas State Board of Agriculture when it was created in 1927 by the consolidation of the Kansas Water Commission and the Division of Irrigation. Section 24-901 of the Kansas General Statutes provides:

24-901. *General plan for water development.* As soon as practicable after organization the commission [now the Division of Water Resources] shall, in conjunction with the federal government by way of securing financial and professional aid and assistance, work out a systematic general plan for the complete development of each watershed in the state, in order to secure the most advantageous adjustment of the interest involved in matters of floods, irrigation, water power and navigation. Where any department of the federal government is now or hereafter may be engaged in the development of plans affecting any of the subjects referred to in this act, this commission may co-operate with such federal department. Water development of all kinds throughout the state shall conform to the general plans adopted by the commission. (Laws of 1917; chapter 172, 4; Revised Statutes 1923, 24-901.)

With the funds made available during the years prior to 1941, the division has maintained and extended the system of stream gaging stations previously established to obtain a continuous record of the quantity of water in Kansas streams.

Neosho River Problems

Since 1941, when funds were first appropriated for the purpose by the

state legislature, the Division of Water Resources of the State Board of Agriculture has been engaged in the preparation of a state plan of water resources development. It is intended that a general comprehensive plan for flood control and water conservation and utilization will be reported for each major drainage basin in the state.

The work done to date has been largely in the Neosho River Basin, the problems of which include floods, insufficient stream flow, pollution and diminishing supplies of satisfactory ground water. Of these, the control of floods is probably the most important. The average direct flood loss is estimated to be approximately \$950,000 annually, with the largest loss resulting from damage to crops and agricultural properties. This figure does not include such indirect losses as loss of business, loss of earnings to farm and industrial employees, loss of income to the communities in and near the flood plain, and loss of life. The major portion of the annual flood loss occurs along the main stream.

In addition to the problem of controlling flood waters, there is the problem of available water supply brought about by the extreme variations in discharge. On an average there flows from the Neosho River basin each year about 1.75 million acre-ft. of water. Although this volume is about a hundred times the combined maximum demand of all the municipalities now using the river as a source of supply, there are times when these communities suffer severely from lack of available water. In 1936 and 1939, the flow in the river at Parsons was zero for a considerable length of time.

Concurrently with a low flow arises the problem of pollution. The river is the natural avenue for disposal of sew-

age and industrial wastes. As the quantity of water in the stream decreases, the concentration of pollution increases rapidly, creating a highly unsatisfactory condition as the discharge approaches zero.

There are ten cities of more than 1,000 population located along the Cottonwood and Neosho Rivers. Most of them are entirely dependent on these streams for their water supply, and all of them use the river as a means of disposing of sewage and industrial wastes. In dry weather, the daily amount of water diverted for municipal purposes was often greater than the quantity naturally flowing in the stream. Under these circumstances, there was obviously a repeated use of the same water as it proceeded downstream. Under the most severe drought conditions, the entire flow of the river was taken through each city water plant, and the quantity returned to the stream through the sewer system then constituted the water supply for the next community below. In addition, the water quality is often impaired by mineral wastes which can be treated only by dilution. For example, oil field brines have a concentration of salts of 40 to 160 times the desirable maximum as established by public health standards. Thus the river becomes wholly undesirable as a source of water supply. In areas of high organic pollution, it fails to support fish and other aquatic life; its value for recreation is destroyed; and it becomes a hazard to the health of individuals and communities located along its course.

Definite evidence of the extent of excessive withdrawal of water from the underground reservoirs of the deep well field of southeastern Kansas is not available, nor is the extent of the

pollution of these sources by mine waters accurately known.

Aside from the problems along the main stream itself, there are other, more local, problems. Flood problems exist on Rock Creek at Burlington, on Labette Creek at Parsons, on Mud Creek at Marion, and on other tributaries. These must be dealt with separately, since they cannot be solved by measures applied to control floods along the main stream. The problem as a whole is complex, and the preparation of a plan to deal with it in the most satisfactory manner to all interests involved is influenced by many factors.

Solution to Problems

It appears that flood conditions can best be rectified through the use of reservoirs, levees and some channel straightening. Low stream flows can be offset by releases from reservoir storage. Pollution can be mitigated through dilution by reservoir releases and a continuation of the policy of more complete sewage treatment by municipalities. The diminishing ground water supplies in the southeastern corner of the basin can be augmented by the diversion of water from the Neosho River into that area.

The four reservoirs recommended by the Corps of Engineers have been adopted as a part of the plan for a flood control system in the Neosho basin, but the inability of reservoirs to control floods completely throughout the basin makes auxiliary works necessary. Levees augmented by some channel straightening appear to be the most logical solution.

The pollution load of the stream is excessive during periods of low flow, particularly in the vicinity of the larger cities. This concentration is expected

to be decreased through the construction of additional sewage disposal works as construction materials become available. A base flow of not less than 50 cfs. can be maintained on the Neosho River below Strawn by the release of water from the conservation pool in that reservoir whenever necessary. This volume of water would dilute the treated sewage load to satisfactory limits.

Most of the municipal and industrial water used in Cherokee and Crawford Counties in the southeastern corner of the basin is pumped from deep wells because the surface supplies are unsatisfactory. In recent years there has been a lowering of the static water level in the well field. The latest census reveals a population increase in this area. Since the demand for ground water apparently exceeds the available supply, it appears that surface sources must soon be exploited. A logical source is the Neosho River. The difference in elevation between the Neosho River bed and the ridge separating the Neosho River from the deep well field is less than 300 ft. The distance between the river and the nearest probable user (Girard) is less than 18 miles. A small storage dam in the Neosho River and a plant to pump water to the area appear to be worthy of consideration.

Proposed Development Plans

A plan of proposed works is presented which, it is believed, offers a basis for the water resources development of the basin. The ultimate plan will be developed progressively, to meet the changing needs of the future (Fig. 1).

The most important unit in the plan is the proposed Strawn Reservoir.

The site selected for the proposed Strawn Dam across the Neosho River is located about a mile and a half upstream from the town of Strawn. The structure will consist of a rolled earth fill 10,650 ft. long with a concrete overflow spillway section 830 ft. long near the center of the dam. The dam would

residents of the valley than any other single unit considered.

The other important reservoirs are upstream from Strawn. They are: Council Grove and Rock Creek on the Upper Neosho, and Marion, Cedar Creek and Diamond Creek in the Cottonwood watershed.

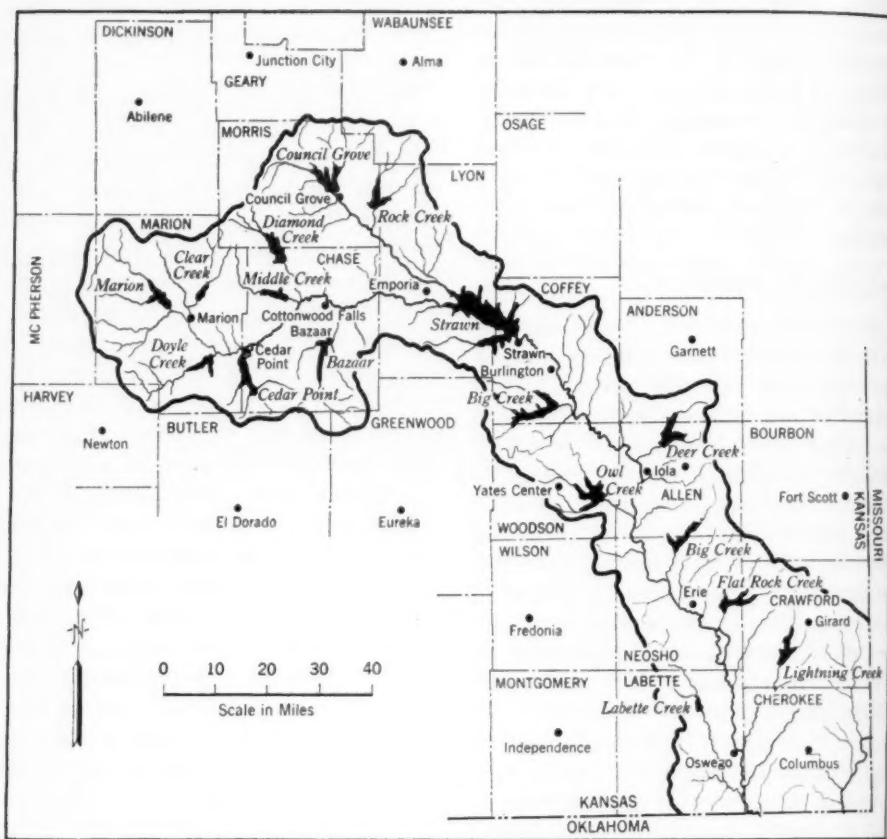


FIG. 1. Neosho River Development Plan

be about 70 ft. high above the stream bed. The total drainage area above the dam site is 2,909 square miles, and the reservoir capacity is 374,000 acre-ft. Due to its size and location, the reservoir will provide the greatest amount of flood protection and assure a dry weather water supply to more

The proposed location for the Council Grove Dam is on the Upper Neosho River, below its junction with Munker's Creek, about a mile upstream from the city of Council Grove. The dam would have a total crest length of 4,120 ft. and a maximum height of 73 ft. above the stream bed. The reser-

voir would have a capacity of 85,000 acre-ft. and control the drainage from 250 square miles.

The location proposed for the Marion Dam on the Cottonwood River is at a site about five miles upstream from the city of Marion and about a mile upstream from the confluence with the South Cottonwood River. The proposed dam would consist of a 7,000-ft. rolled earth fill with a 280-ft. concrete channel spillway.

The Cedar Point Reservoir site is located on Cedar Creek about 3 miles above its confluence with the Cottonwood River. The dam is planned to store 55,000 acre-ft. of water from a drainage area of 121 square miles.

The proposed system of reservoirs would materially reduce flood losses, assure an adequate water supply for all municipalities and industries now in the basin, reduce pollution to a level that would prevent its becoming a nuisance and a menace to health, and provide improved recreation facilities along the stream.

Levees are proposed as a means of augmenting reservoir control. The plan involves the building of levees where needed or desired to provide supplementary protection. The effectiveness of levees can be increased locally through the construction of additional reservoirs on the small tributary streams and by straightening, cleaning and other capacity-increasing improvements on the main streams.

Stream pollution will be reduced by any plan of reservoir operation which provides a supply of water to maintain the low water flows of the Neosho

River at 50 cfs. or more. In addition, pollution of the stream is expected to be reduced materially through the continuation of the present trend towards more complete treatment of municipal and industrial wastes.

The municipal and industrial demands for water from deep wells in Cherokee and Crawford Counties have apparently exceeded the rate of recharge of the producing formations. Relief for this area is available through diversion from the Neosho River. This may be accomplished whenever the need may arise, perhaps by following on a smaller scale the general pattern of the plan developed by the Metropolitan Water District of Southern California. This district, created under California State Law, constructed the Colorado River Aqueduct to bring water from the Colorado River at the eastern boundary of the state to 13 cities in the coastal region, including Los Angeles. By suitable legislation, a water district could be created in southeast Kansas as an agency to bring regulated surplus waters of the Neosho River into areas in the vicinities of Girard, Pittsburg and Columbus. The prospects of continued growth and industrial development of that region, as well as others, may thus be materially enhanced by the orderly development of the water resources of the Neosho River basin.

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Iron and Manganese Removal by Free Residual Chlorination

By **Everett R. Mathews**

A paper presented on March 14, 1947, at the Minnesota Section Meeting, St. Paul, Minn., by Everett R. Mathews, San. Chemist, Div. of San. Eng., State Board of Health, Pierre, S.D.

FOR a long time it has been known that, although chlorine can aid removal of iron and manganese in some waters, in others it seems to have little effect, particularly on the manganese. Furthermore it has been observed that, in waters treated with ammonia and chlorine, manganese is less likely to precipitate out in the distribution system than in waters treated with chlorine alone. As the typical free residual chlorination curve is largely determined by the ammonia content of the water, and as appreciable amounts of free ammonia are naturally present in many iron- and manganese-bearing waters, it seemed possible that free residual chlorination would be effective in removing these minerals.

Studies on this subject were therefore begun by the author in 1940. Considerable work was done in the next two years, until the war interrupted. The studies were resumed in the summer of 1946, and the results thus far obtained may now be presented.

Iron in Water

Whether or not iron will cause difficulty in a water supply depends upon the amount present and its form. Ferrous bicarbonate, in the absence of

organic matter, can usually be oxidized to the insoluble ferric form by simple aeration. Sufficient carbon dioxide is usually released by this process to raise the pH high enough for the oxidation reaction to occur. Removal in this manner is reasonably easy to accomplish, requiring little more than filtration, with or without settling, in addition to the aeration. Iron combined with organic matter, however, is sometimes very difficult to remove.

Manganese in Water

It is now a well-recognized fact that many well supplies in the Middle West and Great Plains Regions contain appreciable quantities of manganese. Also many of the large impounded water supply reservoirs contain manganese in their lower depths. At times these reservoirs contain large amounts of the mineral; at other seasons of the year they are free of it. Similar troubles have been experienced in some of the larger supplies of the country.

Manganese has often been overlooked because of its reluctance to precipitate and because of its delayed appearance in the distribution system. It has been observed that iron usually precipitates quite near the plant and manganese drops out near the center

of the system. Furthermore, manganese may not immediately appear, since its deposition depends upon the formation of manganese dioxide. Once a coating of this oxide is formed, the deposition of additional manganese occurs at a rapid rate and often causes serious difficulties in the system. This accelerated rate of deposition is probably due to catalytic action, although many investigators have attributed it to bacterial activity. The author has observed that, where manganese deposits have occurred, there are usually slime-like bacterial growths on the surface of these formations.

The most commonly employed methods of manganese removal during recent years have included: (a) coagulation at high pH values, followed by filtration; (b) filtration through a base-exchange material adapted to manganese removal; (c) aeration and contact with manganese ores followed by filtration (1); (d) aeration, contact filtration, settling and sand filtration, either with or without chlorination (2, 3). Other methods have been used, but usually they consist of variations of these. Very recently, Edwards and McCall (4) have reported experiences at Baltimore with manganese removal by the use of free residual chlorination.

These methods have usually been successful. It is generally agreed, however, that manganese is difficult to remove, and should be given special attention by the designing chemist or engineer. (Methods which are successful in removing manganese are also successful in removing iron, but the reverse is not true.) A large number of iron removal plants employing aeration, sedimentation, filtration and ordinary chlorination do not remove manganese. Any process whereby these iron removal plants could also

be made to remove the manganese would be of considerable value to the water works industry. It was largely because of this possibility that the author undertook this study.

Experimental Studies

Jar Tests. In order to test the effects of free residual chlorination, it was decided to run jar tests on a number of iron- and manganese-bearing waters. Those selected were the Sioux Falls, S.D., raw water, which contained 4.6 ppm. iron together with 1.6 ppm.

TABLE 1

Mineral Analyses of Raw Waters

	Sioux Falls ppm.	Pierre ppm.
Total Solids	592.0	520.0
Sulfate (SO ₄)	190.0	212.0
Chloride (Cl)	6.0	11.0
Calcium (Ca)	113.0	62.0
Magnesium (Mg)	39.0	30.0
Alkalinity as CaCO ₃		
Phenolphthalein	0	0
Methyl Orange	246.0	154.0
Hardness as CaCO ₃	453.0	281.0
Manganese	1.6	1.6
Total Iron	4.6	0.1
Ammonia Nitrogen as N	0.26	0.25

manganese, and the Pierre, S.D., supply, which contained only 0.1 ppm. iron and 1.6 ppm. manganese. Complete chemical analyses of these waters are shown in Table 1. In addition, tests were made on a number of synthetic samples.

The results of these tests are best shown in Fig. 1-3. It will be noted that these waters show rather typical free residual chlorination curves (Fig. 1), except that the point immediately beyond the break-point shows a somewhat slow rise instead of one paralleling the zero demand curve. It is in this area that the oxidation of the manganese takes place, as shown by

the manganese removal curves (Fig. 2, 3). The iron is oxidized before the manganese.

It was observed that the complete oxidation of the manganese occurred rather slowly, and that the precipitates formed were colloidal and did not settle with any degree of completeness. The important facts observed from these preliminary tests, however, were that the manganese was completely oxidized, so that it could be removed with ordinary filtration, and that the chlorine residuals were sufficiently low so that de-chlorination would not be

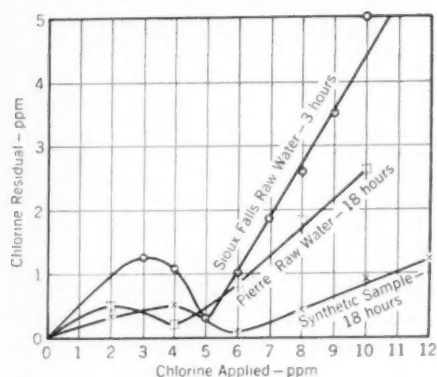


FIG. 1. Residuals From Chlorination of Waters Containing Iron and Manganese

necessary. The synthetic sample, consisting of Pierre water to which 0.5 ppm. of ammonia nitrogen was added, showed that ammonia increased the amount of chlorine required and materially affected the point where the "break" occurred, as well as the oxidation of manganese. It was noted that very little oxidation of manganese took place until all of the ammonia had been removed by the chlorine. This was later borne out by measurements of chlorine and chloramine residuals with a Marks' electrometric titrator (5).

Experimental Plant Tests

The results of the jar tests were sufficiently encouraging to warrant building an experimental plant in which the treatment process could be thoroughly studied and a basis for plant design arrived at. Such a plant was built adjacent to the Sioux Falls iron removal plant, designed for a capacity of approximately 10 gpm. It was put into operation in the spring of 1941 and run continuously for a period of about seven months. The unit consisted of a chlorinator, coke tray aera-

TABLE 2
Removal Without Chlorine*

Samples	Raw	Contact	Settled	Filtered
Chlorine residual, ppm.	0	0	0	0
Iron, ppm.	6.0	2.4	1.6	0.2
Manganese, ppm.	1.6	1.6	1.6	1.1
Free CO ₂ , ppm.	36.0	12.0	14.0	9.0
pH	7.2	7.4	7.4	7.4

* Rate of flow, 10 gpm.; chlorine dosage, 0 ppm.; aerator, contact filter, settling basin and sand filter in service.

tor, contact filter, settling basin and rapid sand filter.

Specifications for these units were:

Chlorinator: Wallace & Tiernan solution feed.

Coke Tray Aerator: 1 × 1 ft. cross section, 4 trays each holding 6 in. of 2-in. coke, trays spaced 16 in. apart.

Contact Filter: 19 in. diameter, 6 ft. high steel shell with filter media consisting of 44 in. of buckwheat anthracite on 12 in. of graded gravel; provisions for backwashing included.

Settling Basin: 8 × 4 × 6 ft., capacity 1,200 gal.

Filter: Conventional rapid sand filter with 30 in. of sand and 18 in. of graded gravel; surface area 5 sq.ft.

Removal Without Chlorination

The effect of progressively increasing chlorine dosages on iron and manganese removal was next studied. As a basis for comparison, however, the plant was first run for a period of two weeks without chlorination. Table 2 shows the results obtained with this treatment.

It is to be noted that, although iron was effectively removed by aeration, contact filtration, sedimentation and filtration, manganese was only slightly

completely removed. No time interval for conditioning the filter sand is required. These results were duplicated many times during this work. A Marks' electrometric titrator was used to differentiate between chlorine and chloramine residuals. These titrations were carried out by A. E. Griffin of the Wallace & Tiernan Co., Newark, N.J.

Removal With Simple Chlorination

In order to determine removals with intermediate doses of chlorine, or doses sufficient to produce only chloramine

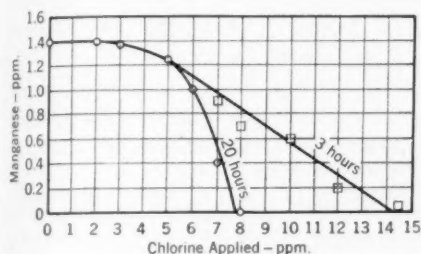


FIG. 2. Relation of Manganese Removal to Chlorination in Sioux Falls Water

affected. This checked the operation of the main iron removal plant and verified the need for additional treatment on this particular supply.

Removal With Free Residual Chlorination

Preliminary tests had shown that the optimum chlorine dosage was in the range of 8 to 10 ppm. A dosage of 9.5 ppm. was therefore applied at the experimental plant, with the results shown in Table 3.

These results show that with the application of enough chlorine to give a true chlorine residual in the filter effluent, the manganese and iron are

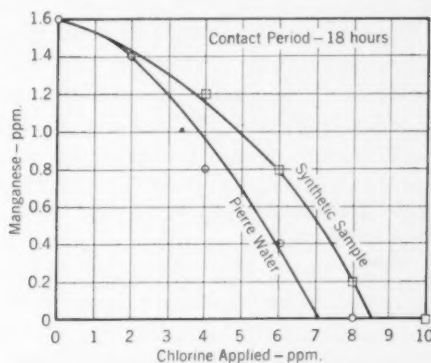


FIG. 3. Relation of Manganese Removal to Chlorination in Pierre and Synthetic Sample Waters

residuals, a dosage of 5 ppm. was used. Results with this treatment are shown in Table 4.

These data show that the manganese is not removed when the chlorine dosage is insufficient to produce a free residual. In other words, so long as the residuals are predominantly chloramine, very little effect is noted on the oxidation of manganese. These experiments were repeated many times and the results were consistent.

From all of the work done on the effect of progressively increasing doses of chlorine it can be stated that: (1)

free residual chlorination (chlorination beyond the break-point) is effective for removal of manganese, (2) simple chlorination (chlorination short of the break-point) has little effect and (3) without chlorination only a small part of the manganese is removed. These data bear out observations which the author has made at other plants, where laboratory data, however, have been lacking. It was noteworthy that, although the jar tests had indicated the oxidation of manganese by free chlorine to be a slow reaction, requiring

and the maximum permissible rate of flow through the sand filter. These values were desired in order to have a sound basis for plant design. It was found that the contact filter removed 56 per cent of the iron and 50 per cent of the manganese. The settling basin removed only 20 per cent of the iron applied and none of the manganese. The sand filter removed the remainder of the iron and manganese applied.

From these data, it seemed likely that the settling basin was not an essential unit in the process. Accord-

TABLE 3

*Removal With Free Residual Chlorination**

Samples	Raw	Contact	Settled	Filtered
	ppm.			
Chlorine residual	0	0.56	0.50	0.46
Chloramine residual	0	0.09	0.09	0.07
Iron	4.5	2.0	1.6	0
Manganese	1.8	1.0	1.0	0

* Rate of flow, 11 gpm.; chlorine dosage, 9.5 ppm.; aerator, contact filter, settling basin and sand filter in service.

several hours, the experimental plant tests showed that it took place quite rapidly. Apparently the surface contact provided by the media in the two filters was responsible for this accelerated action. It was not necessary to have a coating of manganese dioxide built up on the sand grains from long previous usage in order for this accelerated action to take place.

Removal by Various Plant Units

With the chlorine dosage set at the optimum point, a study was made to determine the efficiencies of the separate units in the treatment process

TABLE 4

*Removal With Simple Chlorination**

Samples	Raw	Contact	Settled	Filtered
	ppm.			
Chlorine residual	0	0.05	0	0
Chloramine residual	0	1.11	1.05	1.00
Iron	4.0	3.2	2.8	0
Manganese	1.8	1.6	1.6	1.3

* No free residual. Rate of flow, 13 gpm.; chlorine dosage, 5.0 ppm.; aerator, contact filter, sedimentation basin, sand filter in service.

ingly it was by-passed and the results again noted. Iron and manganese were still completely removed, but filter runs were appreciably shortened, indicating that the detention time provided by the settling basin aided in coagulating the colloidal particles, even though little sedimentation took place. It was evident that, with small amounts of iron and manganese, the need for sedimentation would be slight, but that, with waters high in these materials, provisions for a short detention time would be warranted.

On tests with the contact filter by-passed, it was found that the iron and

manganese could be completely removed so long as sand filtration rates were held below 2 gpm. per sq.ft. At higher rates, significant amounts of both iron and manganese passed through. Also, filter runs were much shorter than when the contact filter was in use.

During this part of the work, a number of observations on design rates of the various units were arrived at. These can be summed up as follows:

Aerator. The coke tray aerator, consisting of 4 trays, each containing 6 in. of coke, removed enough carbon dioxide so that the remainder would not cause corrosion in the distribution system, with rates as high as 15 gpm. per sq.ft. of aerator. Although 2-in. coke was used in the experiment, the actual size of the coke appears to be immaterial. Aeration is not necessary for iron and manganese removal alone, because the chlorine will effectively oxidize these substances even in the presence of a large amount of carbon dioxide. As corrosion would be caused by an excess of carbon dioxide, however, the aeration unit is desirable.

Contact Filter. Even with a high rate of flow, the contact filter was quite effective in removing the heaviest precipitate of iron and some of the manganese, so that the final sand filters would not be overloaded. The contact filter also played an important part in completing the reaction between the chlorine and the ammonia, iron and manganese. Rates of 5 to 8 gpm. per sq.ft. of filter worked out satisfactorily in the experimental units. The buckwheat anthracite was found too coarse for best results. A smaller size anthracite, such as the $\frac{3}{16}$ to $\frac{5}{16}$ in. size Anthrafil, screened and graded especially for use in filters, should be used.

Other filter materials, such as fine coke or gravel, could be used for the contact filter, but Anthrafil possesses properties that are especially desirable for this particular use. The material is light in weight and will therefore backwash somewhat better than gravel. Its surfaces are smooth and do not hold deposits of manganese and iron so tightly that they cannot be washed off.

It is most important that the filter be constructed so that the media can be thoroughly cleaned. For this reason, a down-flow type filter with a backwash system is to be preferred.

Settling Basins. Sedimentation is of little value except that the time interval provided by such a basin causes the colloidal particles to gather together into larger particles, thereby increasing the length of filter runs. This tendency of particles to coagulate is not sufficiently great for any of them to settle out during the 2-hour sedimentation period. For plant design, a small settling basin would be desirable but by no means indispensable.

Rapid Sand Filter. Rapid sand filters of conventional design and conventional rates of 2 gpm. per sq.ft. were found to be most satisfactory. Rates of 3 and 4 gpm. were attempted. The 3-gpm. rate gave a satisfactory effluent, but the 4-gpm. rate was definitely not satisfactory. Studies on the penetration of precipitate into the sand disclosed that, at rates of 2 gpm. per sq.ft., the iron and manganese penetrated the sand to a depth of 12 in. Because of this deep penetration, design rates higher than 2 gpm. are not desirable.

Conclusions

From the large amount of data gathered and from observations on several

plant scale tests, a number of conclusions have been arrived at:

1. Iron and manganese removal can be effected by free residual chlorination at normal pH values and without elaborate treatment facilities.

2. The free ammonia content of the water prevents the oxidation of manganese by chlorine until it is neutralized by the free chlorine residual.

3. Ordinary iron-removal equipment, providing aeration, sedimentation and filtration, plus free residual chlorination, can also be used to remove manganese. Where the amount of manganese is large, the addition of a high-rate contact filter will aid greatly in eliminating the colloidal character of the precipitate and thereby lengthen filter runs.

4. Sedimentation removes very little manganese but does aid in preparing the colloidal precipitate for filtration.

5. The oxidation of manganese by chlorine is a slow reaction when carried out in a settling basin, but proceeds very rapidly when water is filtered through an ordinary rapid sand filter. Furthermore, the filter sand does not need to be coated with oxides of manganese from long previous usage.

6. The chlorine dosage is easily controlled after the proper residual value

for the finished water has once been established.

Acknowledgment

The author wishes to acknowledge the work of the water department of Sioux Falls, S.D., under the direction of Rhea Rees, Supt., in building and jointly operating and studying the experimental iron and manganese removal plant with the State Board of Health. Willard J. Bell, Dist. Mgr., and A. E. Griffin, Asst. Director, Technical Service Div., both of the Wallace & Tiernan Co., Inc., also took part in this study.

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Chlorine Dioxide Use in Plants on the Niagara Border

By Royden N. Aston

A paper presented on April 10, 1947, at the New York Section Meeting, Buffalo, N.Y., by R. N. Aston, Technical Representative, Mathieson Alkali Works, Niagara Falls, N.Y.

CHLORINE dioxide for the treatment of water was first used in water treatment plants on the Niagara border. The research work was carried out at Niagara Falls, N.Y.; the first plant trials took place at Niagara; and the first two plants to adopt the process were located there.

Adoption of Method

Experimental plant operations were started at the No. 2 Niagara Falls water purification plant in January 1944. After working out operational details, full-scale application was begun at the city's main plant in July 1944. The third application of the process in the country was at the Western New York Water Co. plant at Woodlawn, beginning Sept. 8, 1944. Other plants which adopted the process early were those at Greenwood, S.C., Tonawanda, North Tonawanda and Lockport, N.Y., in late October and early November, and Port Colbourne, N.Y., in February 1945.

All of these plants employ the usual method of generating chlorine dioxide, that is, by the reaction of chlorine water with sodium chlorite (1). Water containing at least 500 ppm. chlorine at pH 3.5 or lower is delivered from the chlorinator to a reaction chamber, where sodium chlorite solution is added by means of a proportioning pump. This

yields the chlorine dioxide in an aqueous solution, which is fed to the point of application.

With one exception, the chlorine dioxide solution is fed to the filter effluent at these plants, so that retention time depends upon pumping and clear well capacity. At Tonawanda, however, the chlorine dioxide solution is fed to the suction well just ahead of the high lift pumps, and the retention time is very short. Results here bear out experiences elsewhere in the country, indicating that prolonged retention is not necessary.

The New York plants, with the exception of that at Woodlawn, take their raw water from the Niagara River. This supply is contaminated in varying degrees with sewage and industrial wastes from the cities along the river from South Buffalo and Lackawanna to Niagara Falls itself. The Western New York Water Co. plant takes its water from Lake Erie, which is subject to similar types of pollution.

The most difficult water purification problem of these plants is the suppression of primarily phenolic tastes and odors, at times accompanied by oil wastes. Some algae and vegetation decomposition problems exist in the summer.

Prior to trying chlorine dioxide treatment, the majority of these plants were

using high chlorine doses. They were faced with a major problem in operating under this system, for the taste- and odor-producing substances varied hourly in concentration, making it difficult to feed the proper chlorine dosage. The result was a dose which was either too low to remove tastes and odors, or too high, leading to excessive chlorine residuals and chlorine tastes and odors.

Operations with chlorine dioxide indicated that the optimum method of control was to feed a dose sufficient to handle the majority of taste and odor conditions. It was found that careful control was not necessary, since overdoses produced no taste.

Plant Experiences

It may be instructive to discuss some of the pioneering experiences of the plants along the Niagara border, beginning with the Western New York Water Co. plant at Woodlawn. A somewhat unusual feature of this company's operation is its distribution system. The plant serves the area around Buffalo, utilizing a long distribution system, which extends about 17 miles to the town of North Tonawanda.

Before installing the chlorine dioxide process, the Woodlawn plant was employing super-chlorination. This treatment was not always successful for treating phenols, for, although the water appeared satisfactory when leaving the plant, the phenol taste sometimes returned in the distribution system. This difficulty has been overcome with chlorine dioxide. An average of about $2\frac{1}{2}$ lb. of chlorite per mil.gal. has been fed with satisfactory results. Good control has been established over the taste- and odor-producing substances which, in fall, winter and spring, are predominantly phenols but include some oil wastes. A residual of 0.05 ppm. free

chlorine can be found at the ends of the distribution system. Operation costs are considered comparable to those in the past.

The results of treatment of algae tastes and odors in the summer period have not been all that is desired, but it is hoped, through work at present under way, to obtain better results this summer.

For a time, chlorine dioxide treatment was used at Woodlawn only when tastes and odors were evident in the water. Due to the rapid changes in the character of the raw water, control was difficult and there were occasional complaints. In order to provide an unflinching satisfactory supply, therefore, chlorite is now used continuously. The cost is slightly higher, but it is considered worth the expense to maintain the consumers' confidence and good will.

The Tonawanda plant, as stated, feeds the chlorine dioxide near the suction side of the high lift pumps with satisfactory results. The average dose fed at this plant is about 3 lb. of chlorite per mil.gal., although this is sometimes raised to $3\frac{1}{2}$ lb. per mil.gal.

One aspect of the introduction of chlorine dioxide treatment at Tonawanda merits discussion. When chlorine dioxide application was started and the free chlorine residual began to penetrate the system, satisfactory results were produced at the plant, but some tastes and odors developed in the distribution system, although this condition only persisted for about a week. This development is not unusual, particularly in a plant which has not carried a free chlorine residual in the system before introducing the chlorine dioxide treatment. Apparently certain slime growths in the pipes were oxidized and removed. One plant else-

where in the country actually removed considerable tuberculation from the system by a program of flushing hydrants while using chlorine dioxide.

To avoid such secondary tastes and odors when chlorine dioxide treatment is introduced, it is advisable to start with a low dosage and increase it gradually. After the initial difficulty at Tonawanda was overcome, it was found that a higher chlorine residual could be maintained for greater distances in the distribution system.

During the algae season, copper sulfate is applied to the raw water, and satisfactory taste and odor removal is then accomplished by the use of chlorine dioxide.

At North Tonawanda, continuously satisfactory results have been obtained since the installation of the chlorine dioxide treatment. Generally, 2 lb. of chlorite per mil.gal. has been enough to handle raw water tastes and odors, although at times as much as 3 lb. has been used. On the average, results have been comparable to those at other plants, particularly the neighboring Tonawanda. The effluent from the plant normally has a free chlorine residual of 0.3-0.4 ppm. With this residual, there is no difficulty in carrying a residual of 0.2 ppm. to the end of the distribution system in Martensville, 3 miles away.

Once, for a test run of several days at this plant, prior to the introduction of chlorine dioxide treatment, the water was super-chlorinated so that the effluent had a chlorine residual of 4.0 ppm. During this period there were no complaints of taste and odor from consumers, despite the very high residual and the fact that chlorine dioxide treatment had not yet been instituted. Similar experiences have been cited elsewhere in the country, indicating that,

if a water is free from materials which might produce chlorinous tastes and odors, a true free chlorine residual can be carried at a very high level without tastes and odors. Thus, if it is desirable to carry such a high residual, chlorine dioxide may help eliminate chlorinous tastes and odors, permitting a high residual and yet a very acceptable water.

The Lockport plant intake is very close to those of the Tonawandas. Therefore, to a large extent, the experiences and results have been comparable. The Lockport plant has one fundamental difference, in that the raw water is pre-chlorinated at the intake and has a chlorine retention time in the approximately 20 miles of pipeline. Chlorine is again applied at the filter plant, and chlorine dioxide added at the usual point after filtration. An average dose of about 2.5 lb. of chlorite per mil.gal. is employed. No carbon has been used since the introduction of chlorine dioxide. The plant effluent residual is about 0.28-0.30 ppm. free chlorine.

At Niagara Falls, the No. 1 plant is located above the industrial section, with its intake well out in the river. This plant probably averages in feed about 2-3 lb. per mil.gal. of chlorite in the winter and 3-4 lb. per mil.gal. in the summer, during the algae period.

The No. 2 plant probably has the most difficult problem, as it is located downstream from the industrial plants and takes its raw water from a power diversion canal. Thus, its water comes right off the shore. One day last winter, for example, an extreme phenol condition was experienced. Tests showed that, to obtain a free residual, a dosage of 204 lb. of chlorine per mil.gal. would have been necessary. To handle this extreme condition, it was

found that 28 lb. of chlorite per mil.gal. gave satisfactory results. As this peak demand declined, the chlorite dose was reduced. The reduction probably lagged behind the fall in demand, but there was no trouble due to over-dosage. The ease of control and the ability to get instantaneous reaction has enabled this plant to handle its extreme conditions. Further data on these plants have already been published (2).

Conclusion

From this survey, the results produced by chlorine dioxide treatment may be summarized as follows:

1. Contaminations caused by phenols, along with algae, oil wastes and other taste-producing compounds, have been controlled successfully.
2. A method has been provided whereby severely fluctuating conditions can be handled with ease.
3. A method of taste and odor control has been provided which is economical and does not require careful chemical control.

4. The maintainance of free chlorine residuals throughout the distribution system, without objectionable odors, has been aided.

Chlorine dioxide has been used in solving many types of water works problems. It is not a cure-all in itself for all taste and odor problems, but the results indicate that it is a valuable tool for attacking many water treatment problems.

Acknowledgment

The co-operation and assistance of municipal officials, during plant trials, contributed materially to the development of this treatment. Ralph Bates, Engineer of the State Health Dept., deserves special thanks.

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A Theory of Taste and Odor Reduction by Chlorine Dioxide

By Harry A. Faber

A contribution to the Journal by Harry A. Faber, Research Chemist, The Chlorine Institute, Inc., New York.

A METHOD of water treatment may frequently be used with a considerable degree of success before the theory involved is known. That is, the practice may precede an understanding of the basic principles.

Long before chemists understood the influence of hydrogen ion concentration, the principles of physical chemistry involved in floc formation or the preparation of activated silica sols, the use of alum provided a reasonably successful method of coagulating turbidity in water. Chlorination was demonstrated to be an effective method of reducing the incidence of water-borne diseases prior to our present knowledge either of the bactericidal efficiency of free available and combined available chlorine, or of the ortho-tolidine-arsenite test for determining chlorine residuals.

But a method of water treatment becomes fully successful only when its applications and limitations are understood. What, at present, is known of the applications and limitations of chlorine dioxide as a method for the control of tastes and odor in water supplies?

In his concise summary of experiences with this method of treatment, Royden N. Aston records certain successful applications (1). The use of chlorine dioxide has been demonstrated to control tastes and odors satisfactorily

in six water supplies which are heavily polluted with industrial wastes, principally phenolic in nature. This control has been relatively simple to accomplish.

Why chlorine dioxide treatment is particularly applicable to the treatment of waters containing phenolic pollution, and why the treatment does not require close control, are questions which naturally occur to workers in the field of water purification. The author has frequently found valuable suggestions in a paper by B. A. Adams entitled "Substances Producing Taste in Chlorinated Water" (2). According to Adams:

The product of chlorinating phenol may be *o*-chlorophenol, *p*-chlorophenol or trichlorophenol, although it is more probable that a mixture of these is actually produced. Normally, in the halogenation of phenol the halogen enters the ortho and para positions, but under conditions of extreme dilution it is impossible to predict the resultant product. However, experiments have shown that *o*-chlorophenol gives a distinct taste in a concentration of 1 in 20,000 million, whilst *p*-chlorophenol and trichlorophenol could only be detected in concentrations of 1 in 5,000 million and 1 in 1,000 million respectively; *o*-chlorophenol more nearly resembles iodoform in taste than does *p*-chlorophenol, while trichlorophenol has least resemblance. Iodoform itself gives a distinct taste in a concentration of 1

in 10,000 million, and it differs from *o*-chlorophenol in that it lacks the harsh phenol-like taste of the latter, but this difference can only be detected by experienced observers.

In other words, the particular compound produced by chlorination of a water containing phenol may determine whether or not a noticeable taste will be produced. It is possible that chlorination to the point of producing only combined available chlorine will yield the most readily tasted compound, *o*-chlorophenol; and that chlorination to the point of producing free available chlorine (which has a high oxidation potential) will yield *p*-chlorophenol, one-fourth as readily tasted; or will yield trichlorophenol, one-twentieth as readily tasted.

Chlorine dioxide, which is reported to possess an oxidizing potential two and one-half times that of chlorine, may always react—with no particular regulation of dosage—to produce the least readily tasted trichlorophenol. It is especially probable that this occurs, since the chlorine dioxide method generally used involves: (1) pre-chlorination with small doses of chlorine deliberately to form a chlorophenol (perhaps the ortho compound) and (2) secondary treatment with chlorine di-

oxide to form a tasteless compound (perhaps the trichloro compound). Thus, research may eventually show that chlorine dioxide does not destroy phenols but, instead, converts them to tasteless chloro-compounds.

This interpretation of the action of chlorine dioxide may explain why, as Aston notes, "the results of treatment of algae tastes and odors in the summer period have not been all that is desired." It may be found that no tasteless chloro-compounds can be formed by treating the taste-producing substances released by algae, but that amounts of chlorine dioxide sufficiently large to oxidize the taste-producing substances will be required for satisfactory treatment.

Chlorine dioxide does provide a new tool in the treatment of water. It appears to be especially applicable to the treatment of waters polluted by phenolic wastes. Further investigation of its peculiar properties may disclose its application in a much larger field of taste and odor control.

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An Automatic Residual Chlorine Analyzer

By S. C. Gray and D. V. Moses

A paper presented on Oct. 24, 1946, at the West Virginia Section Meeting, Huntington, W.Va., by S. C. Gray, Chemist, Technical Section, and D. V. Moses, Technical Supt., both of E. I. du Pont de Nemours & Co., Belle, W.Va.

IT is commonly recognized that the chlorine demand of most surface waters may vary appreciably. This factor, combined with frequent operational rate variations in a treatment plant, can considerably complicate the addition of chlorine to water supplies at a rate necessary to secure a residual satisfactory for distribution. The frequent control tests required will absorb a large share of a skilled operator's attention and, at best, will not provide a continuous record. These considerations, which are generally applicable to the water purification unit at the Belle Plant of E. I. du Pont de Nemours and Co., prompted the adaptation of automatic analysis for the measurement of residual chlorine in the treated water. This paper describes the recording analyzer which has been in service for approximately one year at that location.

Previous work on photometric residual chlorine analyzers has been done by Harrington (1) at Montreal, Que., Can., and by Caldwell (2) at Springfield, Ill. The instruments depended on the photoelectric measurement of the intensity of the blue color developed by ortho-tolidine in an alkaline medium. Both analyzers were of the single cell variety, having no compensation for changes in turbidity or color in the water. In addition to the photo-

metric instruments, an amperometric instrument depending on the depolarizing action of chlorine has been described by Baylis (3). In considering these methods, the authors concluded that the measurement of the yellow color produced in the standard acid ortho-tolidine test would present some difficulties, but not insurmountable ones, and that this method would best fit their program and experience. The installation of proper light filters would overcome the variation in tint, and the proper choice of construction materials would eliminate the corrosive action of the acid solution. A double cell compensating arrangement and the appropriate electrical circuit had been worked out for other photoelectric analyzer installations and were available for turbidity correction. With these objects in mind, a recording analyzer was constructed and, after the usual preliminary changes incidental to development work, was put into satisfactory service.

Description of Analyzer

The analyzer consists of four essential parts operating as a unit. In the first part, a controlled quantity of water is heated to 20°C. by a thermostatically regulated immersion heater. In the second part, controlled quantities of water and reagent are mixed in

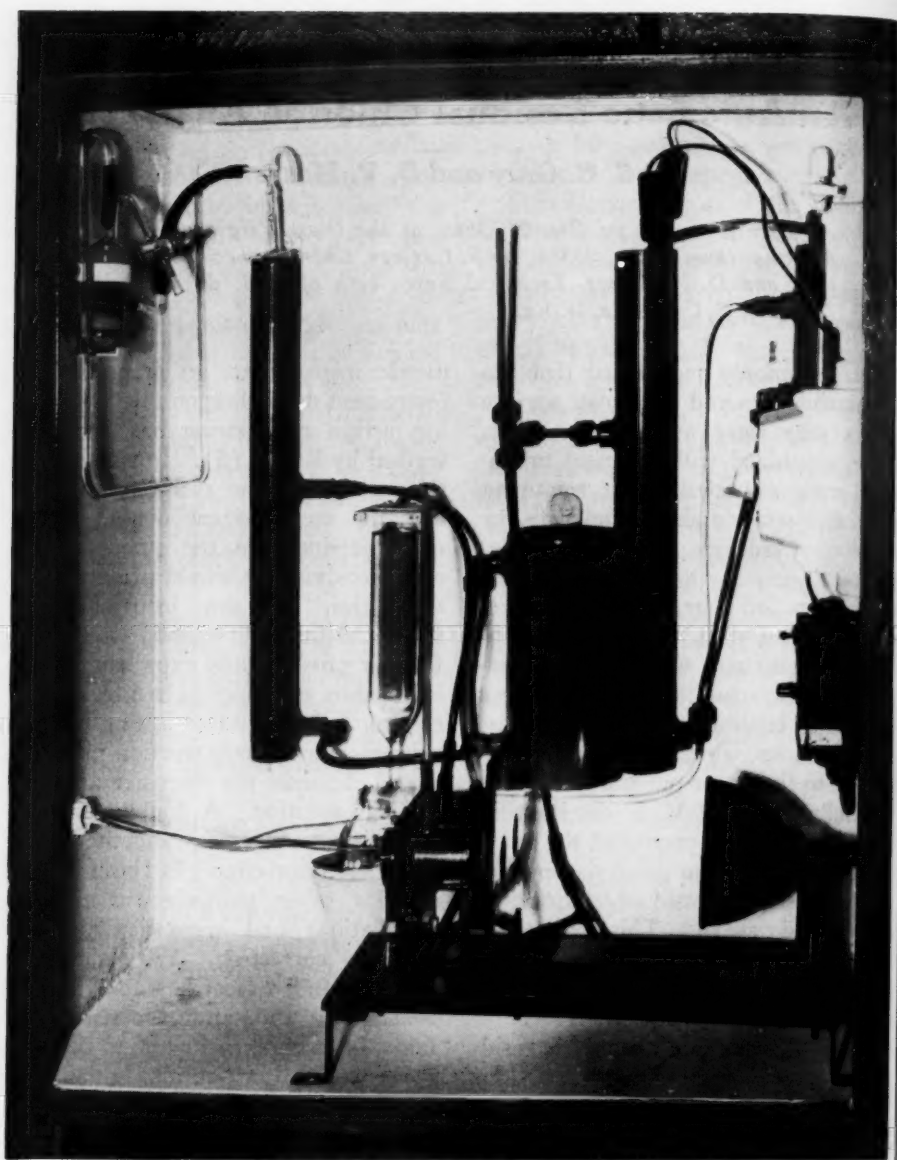


FIG. 1. Analyzer in Operation

proper proportions and retained for a predetermined interval, as required for the development of maximum color. The third part is an assembly of a light system, color filters, solution cells and photoelectric cells. The fourth

part is a commercial recording potentiometer.

Arrangement of the equipment and a flow diagram are shown in Fig. 1 and 2. Relative elevations of vessels are such that no orifice is required to

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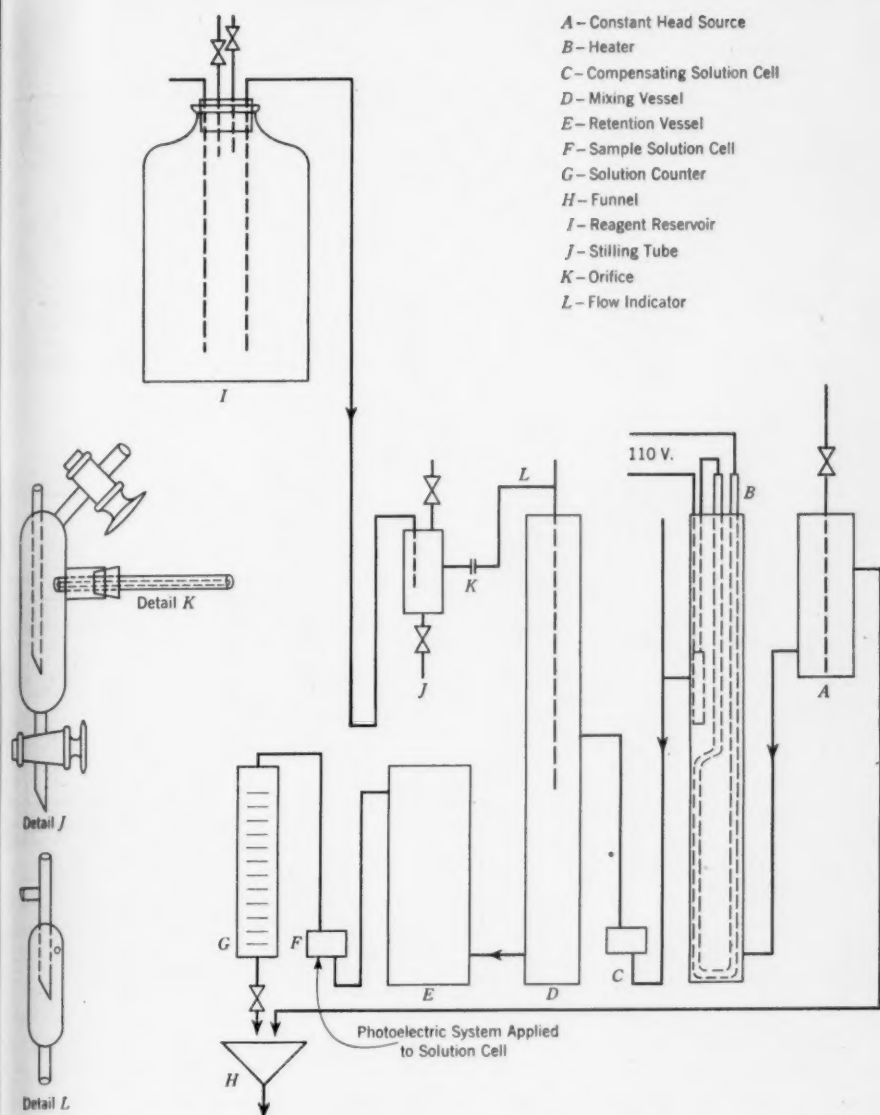


FIG. 2. Equipment Arrangement and Flow Diagram

maintain the desired rate of water flow through the reaction and retention vessels. A 200-watt immersion heater connected with a thermoregulator in vessel B heats the water as required. Reagent reservoir I is a 5-gal. carboy equipped with a submerged capillary

air delivery tube that maintains a uniform static head at the reagent orifice.

Items A-E are of type 316 stainless steel. The solution cells C and F are constructed by clamping Lucite sheets over the ends of 1.5-in. sections of nominal 1-in. pipe, with 1-in. aper-

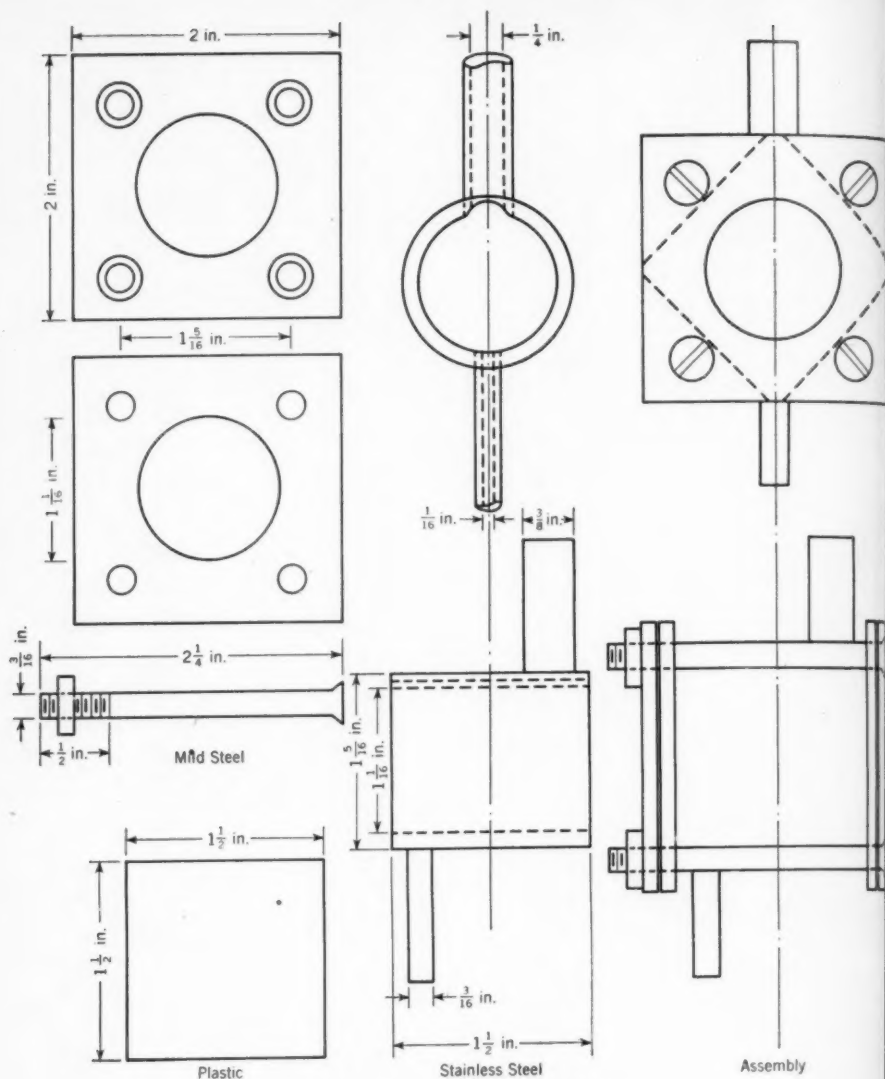


FIG. 3. Details of Solution Cells

tures in the $2 \times 2 \times \frac{1}{8}$ -in. steel tie plates covering the plastic sheets (Fig. 3). The inlet and exit nipples are $\frac{1}{8}$ in. id., and are threaded into opposite ends of the cell bodies. Items *G*, *J*, *K* and *L* are of glass. Orifice *K* is a capillary tube, fire-polished at both ends to constrict the openings to the required diameter. This pro-

cedure produces an orifice that seldom varies in delivery because the thin, flat openings at the tube ends rather than the conventional tapered constrictions minimize the entrance or trapping of foreign matter. All water and reagent lines are made of "Saran" tubing in order to facilitate servicing in industrial use.

Light is supplied by a Bausch and Lomb Reflector Lamp No. 31-33-11-01, equipped with an adjustable transformer (B. & L. No. 31-36-45). Microscope illuminator lamps, Mazda 1493, outside frosted, are satisfactory bulbs for use in the lamp. The parallel position of the solution cells and the use of a null-point circuit eliminate the need for a voltage regulator for the lamp, because variations in light intensity affect both photoelectric cells equally. Light passes through a pair of $50 \times 50 \times 4$ -mm. Corning 503 blue glass filters, then through the solution cells, and impinges on barrier-layer type self-generating photoelectric cells. The photoelectric cells are Photovolt circular elements No. 735, mounted in Model F-3 glass-covered cases.

Leads from the photoelectric cells are connected to a Brown Circular Chart Elektronik Potentiometer, Model No. 151321, with a range of 0-4 mv. Resistors and the standard cell in the potentiometer were disconnected and replaced by a null-point circuit as shown in the wiring diagram, Fig. 4. The standard photocell has three functions: (a) it replaces the original standard cell and balancing resistors, thus producing a more stable circuit and facilitating the adjustment of the zero position of the recorder pointer; (b) it compensates for variations in light due to voltage fluctuations; and (c) it compensates for changes in the color or turbidity of water being analyzed. The residual chlorine range of the analyzer may be increased by decreasing the resistance of R_3 .

Water that has a high iron content may cause deposition of a stain on the windows of the compensating solution cell. This stain will cause a progressive displacement of the zero position of the recorder pointer towards the

lower end of the scale, thus falsely indicating a low residual chlorine value, and requiring frequent resetting of the recorder zero. It may be necessary, therefore, to acidify water of this type before it enters the compensating cell. Acid for this purpose should be obtained by using up to 90 per cent of that specified for the preparation of reagent, so that the total amount of acid added to the water would be unchanged. The acid may be added as a 10 per cent solution in regulated quantities at some point preceding the mixing vessel in Fig. 1, preferably between the heater and the compensating cell. As an alternative, the compensating cell may be flushed periodically with 5 per cent sulfuric acid to remove the deposited stain. No such precaution has been necessary at Belle.

Operation

Operation of the analyzer is automatic and normally requires attention only at 10-day intervals, when the reagent reservoir is refilled. The sequence of operation for putting the analyzer in service may be better understood by referring to Fig. 1.

Water from a header is delivered to the analyzer through a length of tubing. A flow of water through the reaction vessels at the rate of approximately 100 ml. per minute is obtained by adjusting the needle valve in the water line in the cabinet until a continuous overflow from the constant head source *A* is established. Water flows from *A* through heater *B*, compensating solution cell *C*, mixing vessel *D*, retention vessel *E*, sample solution cell *F*, graduated glass solution counter *G*, and to funnel *H*. The rate of water flow is measured by observing the volume collected in the solution counter during one minute. Waste water and

solution fall into a glass funnel supported in a metal funnel and flow to a sewer through a rubber tube.

After the water flow is established, the temperature of water flowing from the sample cell is adjusted to 20°C. by turning on the heater *B* and setting the thermoregulator.

Current to the lamp and recorder is then switched on, and the variable resistor, R_4 in Fig. 4, in the recorder is adjusted until the pointer and pen come to rest slightly above zero.

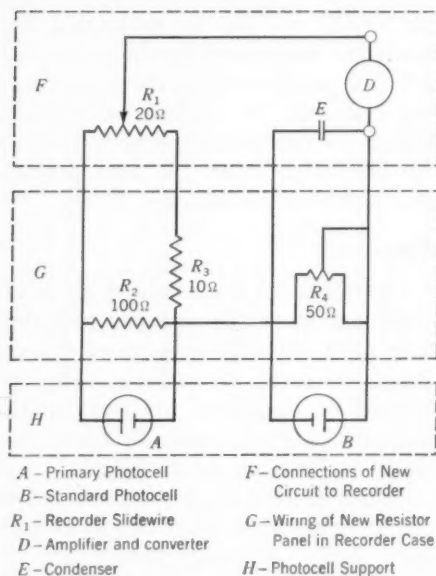


FIG. 4. Wiring Diagram for Brown Recording Potentiometer

Flow of ortho-tolidine reagent to the mixing vessel is started, after the recorder has been balanced at zero, by removing a pinch-clamp from the rubber tube connecting orifice *K* and the flow indicator *L*. Reagent drops by gravity from reservoir *I* and flows through a stilling tube *J* in which occasional particles of foreign matter drop out of the solution stream. Reagent then flows through the orifice at

the rate of approximately 1 ml. per minute, continues through the flow indicator, and finally is introduced below the surface of the water flowing into the mixing vessel. Entrance of displacement air through a tube near the bottom of the closed reagent reservoir maintains a uniform head before the orifice. The accumulation of sufficient foreign matter over the orifice inlet to restrict the flow of reagent may be removed simply by forcing a small amount of reagent backwards through the orifice. This is done by clamping the tube near the flow indicator and then compressing the tube between the clamp and the orifice exit.

Dust should be removed from the lamp reflector, filters and cell windows when required. The zero position of the recorder pointer should be checked occasionally by interrupting the flow of reagent and allowing the recorder pointer to come to rest. Usually it is satisfactory to service the analyzer only when the reagent reservoir is refilled.

Preparation of Reagent

It is convenient to prepare ortho-tolidine reagent in 5-gal. quantities. Technical ortho-tolidine powder contains some insoluble matter, and distilled water may contain particles of foreign matter, both of which would plug the extremely small opening in the orifice. It is necessary, therefore, that all reagent be filtered before it is transferred to the reservoir, and it is preferable that the filtering be done when the reagent is prepared. A satisfactory procedure for making the reagent comprises four steps: (a) 18-20 g. of ortho-tolidine and 100 ml. of 20 per cent hydrochloric acid are ground in a mortar until a thin paste is obtained; (b) the paste is dissolved in 3-4 l. distilled water and filtered

through open textured paper (Whatman No. 4) on a Buchner funnel into a 5-gal. carboy; (c) into the carboy are filtered, in order, 8-10 l. of distilled water, 3.5 l. of 1:1 hydrochloric acid and sufficient distilled water to fill the carboy almost entirely; (d) and, finally, the carboy is rolled until the ingredients are uniformly mixed.

Calibration

The analyzer was calibrated with permanent chlorine color standards containing equivalent quantities of copper sulfate and potassium dichromate. The standard solutions were prepared according to the specifications of *Standard Methods* (4). Figure 5 is a typical calibration chart. The experimental values fall very near a straight line drawn through the origin and indicate that recorder readings for this analyzer are directly proportional to the concentrations of residual chlorine in water. The accuracy of the analyzer in measuring the residual content of water was ascertained in two ways. The analyzer was put into normal operation and the amount of color in the water flowing from the sample cell was measured in a Hellige water test set. Simultaneously with the first test, a sample of the water entering the analyzer was collected, treated with ortho-tolidine reagent, and the color measured in the Hellige test set. Values obtained with the test set agreed closely with recorder calibration values, thus affirming the reliability of both the color-developing and the color-measuring parts of the chlorine analyzer.

Calibration is accomplished by running distilled water and standard solutions through the solution cells. The recorder is balanced at zero with distilled water in both cells. Then, with

distilled water in the blank or compensating cell, a standard solution is run through the sample cell until a constant value is indicated by the recorder. Standard solutions corresponding to at least three points on the recorder chart should be used when a calibration chart is prepared, but a single standard in the normal residual chlorine range is adequate for routine checking of the recorder.

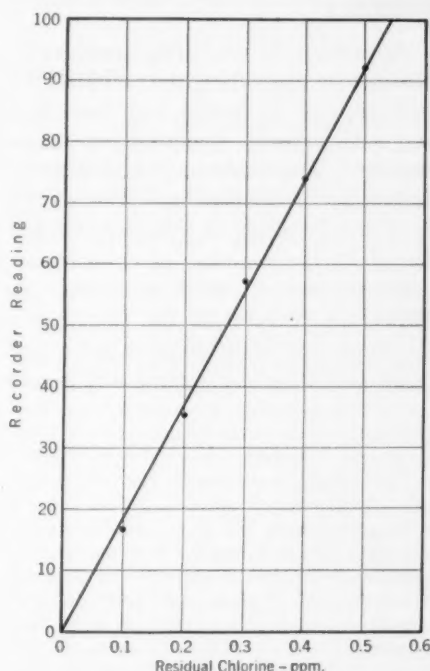


FIG. 5. Typical Calibration Chart

The photoelectric cells are supposed to give uniform performance permanently, unless they are damaged. Nevertheless, three points on the calibration chart should be checked at least once every six months. A wide variation in values for subsequent calibrations will indicate that the characteristics of the cells have changed, and that a new calibration chart must be

prepared. Recalibration is required at any time that a replacement cell is installed.

A routine check of the accuracy of the recorder should be made daily with a portable test set or by comparison with standard solutions prepared according to *Standard Methods* (4). If disagreement is more than ± 0.1 ppm. of residual chlorine, the zero of the recorder should be checked.

Summary

An automatic recording analyzer for continually measuring the residual chlorine content of water has been built and satisfactorily operated for twelve months. The analyzer is based on the principle of measuring photoelectrically the intensity of yellow color produced by the reaction of chlorine and ortho-tolidine in acid solution. An important feature of the analyzer is the use of two solution cells and a null-point potentiometer circuit which com-

pensate for color or turbidity changes in the water and for light variations due to changes in light voltage. A range of 0-0.5 ppm. of residual chlorine was selected for the prevailing conditions, but may be changed as desired by substitution of resistors in the circuit. Accuracy and sensitivity of the analyzer were found to be about ± 1 per cent of the recorder scale for the present range.

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Abstracts of Water Works Literature

Key: In the reference to the publication in which the abstracted article appears, **34: 412** (Mar. '42) indicates volume 34, page 412, issue dated March 1942. If the publication is paged by the issue, **34: 3: 56** (Mar. '42) indicates volume 34, number 3, page 56, issue dated March 1942. Initials following an abstract indicate reproduction, by permission, from periodicals, as follows: *B.H.*—*Bulletin of Hygiene (British)*; *C.A.*—*Chemical Abstracts*; *P.H.E.A.*—*Public Health Engineering Abstracts*; *W.P.R.*—*Water Pollution Research (British)*; *I.M.*—*Institute of Metals (British)*.

HEALTH AND HYGIENE

Control of Dental Caries by Artificial Fluorination of a Water Supply. FRANKLIN M. ERLBACH & EDWIN T. TRACY. Conn. Health Bul., **60: 203** (Sept. '46). In Apr. '45, Conn. State Dept. of Health, in co-operation with administrators and trustees of the Southbury Training School, began treatment of school water supply with sodium fluoride at dosage of 1 ppm. to observe incidence of new dental decay among children in age group from 6 to 16. Present paper, which is a progress report only on first year's experience, lists dental caries experience of 187 fluorine-treated children and of 149 in control group at another state institution. Study of the first year data shows that incidence of new caries in previously sound teeth 30% less in fluorine group. Authors feel it logical to conclude artificially added fluorine in drinking water in low concns. is preventive measure for caries. Measures of effectiveness of such treated waters during entire growth cycle of teeth must await long-range demonstrations now in progress in Grand Rapids, Mich., and Newburgh, N.Y., but it appears to be natural sequence that higher deg. of control would result. Further concluded that there is no evidence to condemn use of sodium fluoride in drinking water under controlled conditions: namely, adequate dental and medical supervision, with advice and assistance of san. engrs. in mech. process.—*P.H.E.A.*

Water-Borne Fluorides and Dental Health. H. TRENDLEY DEAN. Lectures Presented at the Inservice Training Course for Water Works Personnel, (May 22-24, '45), Univ. of Michigan, School of Pub. Health, pp. 100-26. Presence of fluoride (fluoride more than 1

ppm.) in water supplies associated with specific developmental defect of tooth enamel, referred to as endemic dental fluorosis or mottled enamel. Permanent dentition particularly involved. Affected teeth are dull chalky white, may acquire characteristic brown stain (when fluoride more than 2.5 ppm.) or pitted surface (when fluoride more than 4.0 ppm.). Susceptibility of individual teeth requires active calcification of enamel at time of exposure to fluoride-bearing water. Comparatively long-time interval intervenes between operation of causative factor (pre-eruptive) and appearance of defect (posteruptive). Incidence of clinically observable dental fluorosis ranges from approx. 15% at 1 ppm. to 100% above 6 ppm., slight signs occasionally seen at 1.0 ppm. being of no esthetic concern. Race, sex, and color bear no relation. Examn. of 5824 children largely in 12 to 14 yr. age range, in 22 communities with water supplies ranging from 0 to 14 ppm. fluoride, revealed an essentially direct relation between dental fluorosis and fluoride concn. Well-defined geographic boundaries distinguish disease. Control attainable through: (1) abandonment of water supplies contg. excessive fluoride; (2) diln. of waters contg. no more than approx. 2 ppm. of fluoride; (3) casing off aquifers or "plugging back" wells known to be sources of excess fluorides; (4) removal of fluoride from water supplies; (5) institution of efforts to reduce fluoride ingestion, such as high milk consumption, etc. Numerous recent studies of water-borne fluoride and prevalence of tooth decay in school children disclose limited immunity-producing factor in fluoride-contg. waters that is effective whether or not teeth mottled.

Data from 3867 children 12 to 14 yr. old, living in 11 communities where water supplies contain more than 0.5 ppm. fluoride, indicate combined caries experience two to three times higher than that of 3390 comparable children in 10 other towns where water supplies contain 0.5 to 1.4 ppm. fluoride. Small prevalence of decay observed at 1 ppm. fluoride and inverse relation between caries experience and fluoride concn. between 0 and 1 ppm. were very impressive. No signif. advantage in further caries reduction evident beyond 1 ppm. fluoride in water, threshold of mottled enamel. Existing evidence strongly presumes mass control measure against tooth decay. Hypothesis that low fluorination of relatively fluoride-free water supplies constitutes such a measure currently being tested in at least one city (Grand Rapids, Mich.) where fluoride content of water raised to 1 ppm. under rigidly controlled conditions.—*P. H. E. A.*

Experience in the Application of Fluoride to a Public Water Supply. W. LESLIE HARRIS. Lectures Presented at the Inservice Training Course for Water Works Personnel. Univ. of Mich., School of Pub. Health, pp. 148-58. (May 22, 24, '45). Conclusive evidence linking reduction of tooth decay with low-fluoride domestic waters, high regard for water treatment practice, and widespread incidence of dental disease have caused Grand Rapids, Mich., (pop. 170,000), to accept without question application of NaF to public water supply to maint. 1 ppm. fluoride in water delivered to consumer. Limit of 1.2 ppm. arbitrarily established to permit operational fluctuation. Material added to water consists of white free-sifting powder contg. 90-95% NaF and weighing approx. 75 lb./cu.ft. Insoluble fraction 1.2%. Except for encrustation in soln. lines, ideal for controlled application. Less dense grades rejected to maint. dust at min. Amt. currently required about 12,000 lb. monthly; avg. cost \$1.43 per mg., yearly expenditure of \$11,000, or 6½¢ per person supplied. Application is made to influent of filters (effluent of settling basin). Here it is possible to measure accurately water flow under all load conditions, avoid removal of fluorides by alum coagulation (pretreatment), and elim. tech. difficulties imposed by operating facilities in post treatment. Physical capac. of (dry gravimetric) feeder limits max. possible concn. to 10 ppm. fluoride, which nontoxic in occasional short exposures. Actually signif. dosage above 1

ppm. fluoride unlikely. Daily tests for fluoride by modified method of Scott made of raw, settled and filtered waters, and water samples from 5 points on distr. system. Weekly tests also made of water samples from 8 fire houses at representative locations. Operation satisfactory. No taste transmitted to water by addn. of 1-1.2 ppm. fluoride as NaF.—*Ed.*

Dental Caries Experience in Relocated Children Exposed to Water Containing Fluorine. Incidence of New Caries After 2 Years of Exposure Among Previously Caries-Free Permanent Teeth. H. KLEIN. U.S. Pub. Health Repts. 60: 1462 ('45). In autumn of '42, 316 children of Japanese ancestry, who were of school age, were removed from Los Angeles; 120 of them went to center in California where water supply contained 0.1 ppm. fluorine, and 196 went to center in Arizona where water supply contained 3 ppm. fluorine. In summers of '43 and '45 teeth of children examd. In '43 numbers of permanent teeth free from caries differed little in 2 groups of children. In '45, among children who had been aged 8 to 10 in '43, number of teeth attacked by caries since '43, expressed as percentage of permanent teeth unaffected in '43, appreciably less in Arizona than in California. Among older children incidence of dental caries could not be correlated with consumption of fluorine.—*W. P. R.*

An Inquiry Into the Public Health Hazard of Sewage Disposal From Railway Conveyances. Kenneth F. Maxcy. Sanitation Research Project, Assn. of Am. Railroads (Nov. 8, '46) 17 pp. Idea that track poln. might be responsible for dissemination of micro-organisms causing enteric infections and disease—especially typhoid fever—first considered in U.S. about turn of century. Roadbed then called "natural culture medium" for typhoid. Poln. of watersheds providing community supplies feared, and prevented only by locking doors of water-closets, a practice disagreeable and even unhealthful to passengers. Need for some receptacle to hold wastes without causing nuisance until safe dischg. possible pointed out, but today still unfilled. In 1920 Std. San. Ry. Code formulated and accepted by Am. Ry. Assn. and 45 states. Studies recently made of typhoid incidence for 1900-20 to test tracks as source of infection. Exhaustive research showed no prev-

alence of disease among persons living near rys. or terminals. Ry. employees also studied, with negative, but less conclusive results. Epidemics resulting from watershed contam. likewise not definitely traceable to rys. Longevity and transmission of *Eberthella typhosus* in roadbeds now considered dubious. Public water supplies exposed to contam. from numerous sources, of which rys. only one, and best protection for them is chlorination. Considered that original hypothesis of contam. from rys. ought to be modified. Nuisance possible at times and under certain conditions, but menace not statistically observable. Possibility of danger below this threshold of measurement remains, but even this can be reduced to insignificance by tech. improvements now possible. 115 refs.—*Ed.*

Cholera Situation, Summer 1946. KNUD STOWMAN. *Epid. Inf. Bul. UNRRA*. 2: 674 (Aug. 31, '46). Cholera spread rapidly in July throughout large part of China. Shanghai seems to be chief center, persistent Canton epidemic rating second in importance. Particular attention being given in Shanghai to betterment of water supply by closure of wells, chlorination and the provision of new water points by opening of city water hydrants. There has been little chlorination of well water in Canton, where disease has been epidemic since March. Cholera reported in Manchuria, Hong Kong, Siam, India, and other localities in varying proportions. No case of cholera west of India since '31, apart from Afghanistan-Iran outbreak in '38-'39. In year of economic and political disturbances, which is moreover distinctly bad cholera year both in China and in India, blocking of any westward inroad indeed no mean achievement.—*P. H. E. A.*

Inactivation of the Virus of Infectious Hepatitis in Drinking Water. JOHN F. NEEFE, JAMES B. BATY, JOHN G. REINHOLD & JOSEPH STOKES JR. *Am. J. Pub. Health* 37: 365 (Apr. '47). Virus of infectious hepatitis excreted by infected persons. Some communities use pold. streams as original source of supply. Appears possible that survival of hepatitis virus occurs in supplies inadequately treated contributing to spread of this disease. Experiments with human volunteers conducted with variously treated waters. Control contam. with suspensions of hepatitis virus and *Esch. coli* in distd. water. Treatment included coagulation with 34.24 ppm.

sodium carbonate and 68.5 ppm. aluminum sulfate, settling, filtration through diatomite filter, chlorination and de-chlorination after 30 min. with sodium sulfite. Chlorine dosages of 3.25, 7.5 and 15.0 ppm. used. Activated carbon added with coagulants in 3 samples. Min. chlorine dose used produced 30 min. residuals of 1.1 ppm. total and 0.4 ppm. free chlorine which was sufficient to inactivate virus in treated waters. Same dose in contam. water not pretreated by coagulation, settling and filtration did not inactivate hepatitis virus. Coagulation, settling and filtration without chlorination did not eliminate virus effect as 40% of volunteers developed disease; but resulted in prolongation of incubation period and 40% decrease in incidence as compared to controls. Min. effective dose not detd. due to lack of volunteers.—*F. J. Maier.*

Hematuria Due to Picric Acid Poisoning at a Naval Anchorage in Japan. A. H. HARRIS, O. F. BINKLEY & B. M. CHENOWETH JR. *Am. J. Pub. Health*, 36: 7: 727 (July '46). 2 persons on board ship in naval anchorage near Wakayama found to have microscopic haematuria. Discovery made while these patients being treated for "irrelevant" conditions. Medical officer then examd. urine of 245 men, almost whole ship's company, and found that all had asymptomatic microscopic haematuria. Investigation by the Naval Epidemiology Unit then revealed that haematuria widespread. Some of personnel of 28 ships examd., and cases of haematuria found on 25 vessels. Out of total of 383 individual specimens examd., 138 found to show red blood cells. No evidence of haematuria reported among military personnel ashore. In most, haematuria trifling; in only 3 cases was there evidence of involvement of kidneys. Investigation indicated that haematuria probably due to chem. contam. of drinking water distd. from sea water in harbor. Aromatic amines or nitro-compds. suspected, and information that picric acid had been dumped in the sea somewhere in general area directed special attention to this compd. Chemical inquiry, of which some details are given, demonstrated presence of picric acid in some samples of (1) distd. water, (2) scale deposited in evaporators, (3) scale in distributing pipes, and (4) urine. Confiscated Japanese picric acid seems to have caused poisoning. More than 100 tons of this substance, in powd. form, had been dumped, over period of 6 weeks, in

sea about 15 mi. from harbor. Would appear that prevailing wind and tide carried acid, before it had become completely dissolved and dispersed, to anchorage where ships were pumping harbor water into stills. In general, but with certain puzzling exceptions, ships that were significantly affected were those which had distd. water at Wakayama while dumping in progress.—*B. H.*

Methemoglobinemia Occurring in Infants Fed Milk Diluted With Well Water of High Nitrate Content. ROBERT L. FAUCETT & HERBERT C. MILLER. *J. of Pediatrics*. 29: 593 (Nov. '46). Case histories of 3 infants having methemoglobinemia described. All less than 4 wk. of age and had been fed milk mixts. contg. well water. Nitrate content of water varied between 70 and 300 ppm. with traces of nitrates. All cases recovered; one spontaneously and two (twins) after injections of methylene blue. Methemoglobinemia apparently develops as result of amt. of nitrates ingested in relation to body wt. Adults using water causing methemoglobinemia in infants had not been affected. Six-months-old infant living in same house as 2 of cases described and fed same water since birth also not affected; his formula made of equal parts of whole milk and well water, whereas twins fed mixt. contg. one part Similac and two parts boiled water. Appears likely that methemoglobinemia occurs over wider area than has been reported because location and condition of wells dug in rural areas may be such as to permit contam. by nitrogenous compds. It would also appear that infants living on farms should be fed milk mixts. which are relatively concd. or made with water from source known to be satisfactory.—*F. J. Maier.*

Present Position of DDT in the Control of Insects of Medical Importance. FRED C. BISHOPP. *Am. J. Pub. Health*, 36: 593 (June '46). DDT has not cured all ills involving insects but many of early predictions regarding efficacy have come true. Vast amt. of information on toxicology and pharmacology of DDT now available. May be summarized: (1) DDT toxic to all higher animals, but acute toxicity less than some of common arsenical and nicotine insecticides; (2) median lethal dose varies widely with species and individual, ranging (in mg./kg. of body wt.) from 150–250 for mice, 300–500 for rabbits, over 200 for monkeys, over 1300 for chickens

and over 300 for horses. For comparison, median lethal dose of arsenic trioxide for rabbits 15–30, sodium arsenite about 50, basic lead arsenate—180, sodium fluosilicate—120; (3) daily ingestion of doses much below lethal dose may cause death before acute lethal dose reached; (4) powd. DDT applied to skin not toxic; (5) oil solns. of DDT, when absorbed through skin cause toxic effects; (6) DDT not promptly and completely elimd. when ingested or absorbed is stored in tissues (especially fat), and for more than lethal dose may be stored with no apparent ill effect on animal; (7) if ingested in considerable quantity, DDT excreted in milk; (8) DDT a nerve poison; (9) shows little or no sensitizing action; (10) *where used as recommended* for control of human parasites and household insects, DDT insecticides not harmful to human health—this includes DDT in aerosols and several other forms in which recommended. DDT *must not* be allowed to get into food or to be ingested accidentally. Prolonged exposure of skin of 2.5–5% oil emulsions should be avoided, those contg. 25% or more DDT should be removed immediately with soap and water. No known antidote exists; if ingested by man, physician should be called. Medical attention usually consists of stomach lavage followed by saline cathartic. One of outstanding DDT characteristics is persistence. Care therefore necessary when using on food or feed crops and products. Tentative figure of 7 ppm. set for residue on fruits and vegetables (same as for lead and fluorine). Bureau Entomology and Plant Quarantine does not recommend use on cabbage and similar vegetables after head or other edible parts formed; also not recommended on alfalfa, corn, crops fed to stock—especially dairy animals. DDT so effective against many troublesome agricultural pests that toxicity of DDT residues must be definitely detd. as soon as possible. **Effect on Plants.** DDT very effective in killing Japanese beetle larvae in soil but even the 25 lb./acre required affects growth of bush beans, soybeans, rye. In general plants not harmed by DDT applied to them; but injury, especially to young plants, suffered by squash, cucumbers and other cucurbits treated with dust or spray. **Effect on Beneficial Life.** Oil solns. of DDT applied by airplane at 5 lb./acre rate to forest areas while birds are nesting may destroy large percentage of birds—2 lb./acre or less harmless to birds. Some fish killed at 1 or 2 lb./per acre, few at

low dose of $\frac{1}{2}$ lb./acre, with emulsions more destructive than oil solns. Anopheline larvae killed by $\frac{1}{16}$ lb./acre or less, in lab. 0.01 ppm. found effective. Honey bee not extremely susceptible to DDT; some insect friends as lacewings, wasp and fly parasites of pest insects more susceptible than some insect foes. If formulation and time and method of application carefully chosen, min. dosages and number of applications used, and then used only for pest on which DDT known to be effective, author confidently believes no serious consequences will result from use of DDT. Probably greatest hazard is use over large area without guidance. DDT remarkably effective against such pestiferous and disease-carrying insects as mosquitoes, flies, fleas, bedbugs and sand flies. Cockroaches and ticks and a number of other household pests yield reasonably well to DDT if properly applied. Rapid increase in knowledge of material testified to in 965 technical articles published between Jan. 1, '43 and June 30, '45. *Status of Supply.* In later months of war, production reached 30 mil.lb./mo., production capac. apparently exceeds present demand so no shortage anticipated. Employed against insects, mainly in 4 forms: powds, or dusts, solns., emulsions and suspensions. Effect of DDT influenced by kind of applicator used, amt. applied, temp., exposure of treated surface to sunlight, wind, rain, dust and oil vapors and other factors. *Distribution Methods.* 5-10% dusts can be applied with reasonable satisfaction by bellows, shaker cans, dusters, rotary hand dusters, various power dusters and by airplane. Among new developments are new power sprayers with high velocity blowers. Wind veloc. of 100-150 mph. or more obtained. Use of this equip. results in great saving of time and materials; e.g., 1 pint of insecticide may cover tree where 15-20 gal. formerly required. If used as residual spray on surfaces, crawling insects killed 3-12 mo. after spraying. Usual quant. of DDT is 200 mg./sq.ft. of surface, provided 1 gal. 5% soln. or suspension applied to 1000 sq.ft. Water-dispersible DDT advised in barns, in stores or dwellings; 5% soln. in odorless kerosene preferable. Aerosols of greatest value for use against mosquitoes, flies and other free-flying insects in enclosed spaces; may be used for temporary protection out of doors where there is little air movement. (Aerosol is suspension of fine particles in air or gas, as in a fog or mist.) DDT acts as both contact and stomach

poison, but is particularly effective as contact killer. Is not rapid in its effects—usually $\frac{1}{2}$ hr. or more required. Effect on mosquitoes, flies, fleas, filter or moth flies, bedbugs, lice, cockroaches, and mites thoroughly discussed in article which should be referred to for specific information.—*Martin E. Flentje.*

Toxicity of DDT to Man. F. M. G. STAMMERS & F. G. S. WHITEFIELD, *Nature* (Br.) 658 (May 18, '46). At Royal Naval School of Tropical Hygiene in Colombo, team of 15 men (one Tamil and 14 Sinhalese) employed continuously for 5 to 7 mo. in prepg. and spraying kerosene soln. of DDT (5%). On avg., 24 hr./wk. actually spent in spraying. At beginning, protective clothing and gauze masks issued, but owing to heat, men refused to wear these protective devices. Most of spraying done in confined spaces and men often exposed to splashes and drip from ceilings, as well as to leakages from sprayers (which received rather hard treatment). After day's operations, white frost of DDT crystals could be seen on exposed portions of skin, while overalls frequently satd. Despite this very considerable deg. of exposure to DDT, none of men developed signs or symptoms of intoxication. Each man questioned about general fitness and received clinical examn. Liver function estd. by oral hippuric acid synthesis test, and hemoglobin estns., red- and white-cell counts, including differential white-cell counts made; examn. of urine for abnormal constituents and stools for parasitic ova, worms and cysts also carried out. Similar examns. made on control group of men. No evidence of ill health due to DDT detected.—*B.H.*

"DDT" Poisoning in Man. I. M. MACKERRAS & R. F. K. WEST, *Med. J. (Australia)* 1: 12: 400 (Mar. '23, '46). Following observations on DDT poisoning made in New Guinea: (1) *Poisoning by ingestion.* Native cook-boy used DDT in mistake for baking powder to make a tart, which was eaten by some 25 soldiers. All suffered from feeling of giddiness and weakness, commencing from 1-2 $\frac{1}{2}$ hr. after meal. Four vomited and two taken to field ambulance. All recovered within 48 hr. (2) *Poisoning by contact.* Malaria-control duty man, grown careless, allowed his right hand to become covered with 5% DDT soln. in light diesel oil, on 6 days out of 7 for some time (probably weeks). Finally he developed numbness, muscular weakness

and some swelling of hand, and at same time suffered from severe deep-seated headache, not relieved by aspirin. About fortnight after onset of these symptoms, he began to vomit at night and ran temp. of 101°F. No indication of liver involvement. At this time he "reported sick" and was put off duty. Swelling, numbness and headache disappeared in 4 days, but it was 14 days before muscular power was restored to normal. In another case, a man measuring DDT dropped the measure into the tin and a cloud of powder flew out. [This must have been a special

prepn.: ordinary conc. heavy and rather sticky.] Some of it got into his eyes causing intense pain for 4 days, requiring repeated injections of morphine and application of castor oil drops; he was blind for a fortnight and had severe headache for same period. Recovery complete. These cases indicate need for care in handling DDT concs.; but they suggest that if contact with DDT stopped with advent of symptoms (numbness, weakness, headache) recovery will be complete without any treatment other than symptomatic treatment.—B.H.

BACTERIOLOGY

Study of a Method of Concentrating Water Samples for Bacteriological Investigations.

OSVALDO A. PESO. Rev. Admin. Nacional Agua (Arg.) 20:115:54 (Jan. '47). Aluminum hydroxide adsorption method of Dienert and Guillard, modification of Wilson's pptn. method, investigated in order to det. its suitability for extracting pathogenic organisms from samples of natural and treated waters. Advantage of method selected over that of Wilson is that addn. of previously prepd. aluminum hydroxide to contamd. water, instead of floc production in sample by addn. of aluminum sulfate soln., does not change original characteristics of water during concn. process. Results obtained with distr. system samples which had been de-chlorinated and experimentally contamd. with *Salmonella typhi*, *S. typhimurium*, and *S. dysenteriae* types Flexner V and Boyd 2, indicate method capable of recovering bacteria without affecting their vitality. Equally satisfactory results obtained with water from Plata R. experimentally contamd. with above-mentioned organisms. Although recoveries up to 90% obtained by direct plating of floc retrieved from samples contg. from 5 to 10 bacteria per l., recommended that ppts. obtained from samples suspected to have less than 50 organisms per l. be transferred to enrichment media prior to plating for isolation of organisms. Desoxycholate-citrate liq. medium used for enrichment believed most suitable for development of intestinal pathogens.—J. M. Sanchis.

A Superior Culture Medium for the Enumeration and Differentiation of Coliforms.

GEORGE H. CHAPMAN. J. Bact. 53:504 (Apr.

'47). Medium, based on Pollard's finding (Science, 103:758), composed of: water, 1,000 ml.; Difco yeast extract, 3 g.; proteose No. 3 peptone, 5 g.; lactose, 10 g.; agar, 15 g. Reaction adjusted to pH 6.9, then 0.1 ml. tergitol-7 and 2.5 ml. 1% bromthymol blue added. When incubated 20 hrs. at 37°C., *Escherichia* produces yellow colonies surrounded by yellow zones. *Aerobacter* forms greenish yellow "gum drop" colonies, larger than those of *Esch.* and usually surrounded by yellow zones. *Paracoli* and other lactose non-fermenters give colonies usually surrounded by blue zones. A few strains of *Neisseria catarrhalis* grow but develop minute, rough, blue colonies with blue zones. No other bact. observed. Growths do not "run" as with other freshly prepd. media, even *Proteus* tending to spread less. Apparently coliforms not inhibited, thus permitting recoveries from minute inocula. Counts about 30% higher than on other selective media.—Ralph E. Noble.

Investigation of a Liquid Enrichment Medium for Bacteria of the *Shigella* Group.

RAMON H. LEIGUARDA. Rev. Admin. Nacional Agua (Arg.) 20:115:13 (Jan. '47). Desirability of effective enrichment medium for investigation of *Shigella* organisms in water and sewage prompted investigation of liq. medium similar in compn. to Leifson's desoxycholate-citrate agar. Aside from elmn. of agar, liq. medium differs from that of Leifson's in that it contains no lactose nor lead chloride. Lactose elimd. to prevent rapid lowering of pH as result of fermentation. At pH below 7.5 desoxycholate ppts. removing neutral red, thus losing essential components of medium.

Final pH adjusted to 7.6. Parallel expts. showed absence of lactose did not diminish sensitivity. Since purpose of lead salt in solid medium is detection of hydrogen sulfide producing organisms, its inclusion in liq. media for enrichment of *Shigella* unnecessary. With exception of types Sonne and Shiga, liq. medium showed marked sensitivity for detection of *Shigella* organisms in suspensions prepd. with pure cultures. Sensitivity retained in presence of large nos. of *Esch. coli*, *Proteus vulgaris*, and *Pseudomonas aeruginosa* obtained from pure cultures. Some difficulty experienced in recovering *Shigella* from suspensions prepd. with heavily contamd. river water. Organisms of Flexner and Boyd types recovered but none of Sonne type. Larger no. of recoveries obtained after 24 hr. incubation than after 48 hr. Liq. media not effective in recovering *Shigella* from experimentally prepd. suspensions in sewage. Although little work done with *Salmonella*, desoxycholate-citrate liq. media appears suitable for their development. No difficulty experienced recovering *S. typhi* from contamd. river water. Although less easily, *Salmonella* also recovered from sewage suspensions. During these expts., *S. kentucky* unexpectedly isolated from 3 different samples of sewage.—J. M. Sanchis.

Proposed Changes in Incubation Temperatures for Standard Agar Plate Counts. JAMES D. BREW. J. Bact. 53:372 (Mar. '47). [Author emphasizes principles equally important in bact. water anal.] Recently, milk control lab. workers observed greater variabilities in ests. made at 37°C. than at lower temps. Apparently, 37°C. is close to or possibly above max. growth range for some bact.; also temps. in different incubators vary more widely in 37°C. range than at lower temps. Not only may incubator temps. vary according to make but also at different points inside each. Some 37°C. incubators may run as high as 45°C., sufficiently warm to inhibit growth or possibly kill some orgs. Proposed to lower incubation temp. to 32°C. for 48 hrs. Workers found variability in ests. at 32°C. and 37°C. about 4% and 25% respectively. In addn., total no. estd. at 37°C. avgs. about 50% lower than those at 32°C.—Ralph E. Noble.

Bacteriological Examination of Water Using Silicate Media. W. OLSZEWSKI & I. RATHGEGER. Gesundh.-Ing. (Ger.), 67:133 ('44). Use of silicic acid as substitute for agar and

gelatin in culture media employed for bact. examn. of water has proved so satisfactory that it is likely to be used considerably in future. Authors suggest that counts obtained on silicate medium should be distinguished by being called silicate counts. They have developed simplified method of examn. which is suitable for use in field, and which includes taking total bact. counts at 22° and 37°C., coliform count on Gessner silicate medium, coliform count and count of indol-formers at 37°C. using inoculation into lactose-silicate medium, and coliform count at 45°C. Details given of methods of preparing medium and of carrying out tests.—W.P.R.

Study of Merits of Various Confirmatory Media [for Coliform Bacteria]. P. J. PHILSON. W. W. Eng., 99:386 ('46). Confirmatory tests run on all of 330 pos. presumptives obtained on Columbia city water in '45. In each case confirmatory tests run in following liquid media: (1) B.G.B., (2) formate ricinoleate, (3) fuchsin, and (4) crystal violet. During Jan. and Feb. E.M.B. agar plates also streaked. Tabular data reporting monthly totals show very wide variations in results obtained with 4 liquid media, in some instances variation amtg. to as much as 700%. B.G.B. found most highly selective of media tested.—C.A.

Comparative Studies of Enterococci and *Escherichia coli* as Indices of Pollution. MORRIS OSTROLENK, NORMAN KRAMER, & ROBERT C. CLEVERDON. J. Bact. 53:197 (Feb. '47). With positive correlation between insanitary methods of food production and presence of *Escherichia coli* in finished product, presence also of confirmed fecal coliform strains interpretable as having san. signif. If environmental factors tend to permit or promote multiplication of normal flora, as well as coliform contaminants, however, interpretation of san. signif. of *Esch. coli* becomes involved. Not improbable that technical difficulties hitherto prevented investigation of intestinal micro-organisms other than *Esch. coli*. Authors describe method of isolating fecal streptococci at 45.5°C. in 0.05% sodium azide ("SF") broth as primary enrichment, followed by streaking on same medium solidified by 1.5% "SF" agar, and incubating at same temp. *Strep. fecalis* or *Strep. liquefaciens* predominated in 531 cultures from nut meats, frozen fruits, berries, vegetables and freshly packed crab meat by

this method. Enterococci in artificially contamd. soils and pecan meats, and in normal feces, appear to survive longer than *Esch. coli* under identical storage conditions. The adverse effect of high total bact. counts on isolation of this org. was not experienced with "SF" media for isolating enterococci. Authors believe value of fecal strep. enhanced as index of poln. Data from food-producing establishments show excellent correlation between sanitation and recovery both of *Esch. coli* and enterococci. [This article of interest in relation to work by Mallman *et al* on "A Comparative Study of Chlorine and Bromine for Swimming Pool Disinfection." T. L. Vandervelde, W. L. Mallman and A. V. Moore. Presented at the Eng. Sec., A.P.H.A. meeting in Cleveland, Nov. '46].—*Ralph E. Noble.*

The Distribution of Coliform Bacteria in Human Feces. JOSE C. KEMPNY. Rev. Administracion Nac. Agua (Arg.) 10:103:15 (Jan. '46). Since estn. of signif. of coliform organisms isolated in course of routine bact. examn. of water depends on knowledge of relative frequency of coliform types in human feces, which may vary in different parts of world, these results, which were obtained by differential methods recommended in Rept. No. 71 ("The Bacteriological Examination of Water Supplies") of the Ministry of Health, are worth recording. Strains, 1000 in number, isolated on eosin-methylene blue and litmus-lactose agar from 100 specimens from adults in Buenos Aires. Strains classified as follows:

<i>Esch. coli</i> , type I, fecal.	90.7%
Irregular, types I and II.	5.5
<i>Aer. aerogenes</i> , type I.	0.6
<i>Aer. aerogenes</i> , type II.	0.4
Irregular, type VI.	2.7
<i>Aer. cloacae</i>	0.1

Tests showed *Esch. coli* to be present alone in 86% of specimens, and along with *Aer. aerogenes* in 13%. In whole series there was complete negative correlation between V.P. and M.R. tests, and 97.3% of strains produced acid and gas in MacConkey broth at 44°C. Strains other than *Esch. coli* Type I, which produced acid and gas in MacConkey broth at 44°C. belonged to irregular types II and VI.—*B.H.*

Estimating the Age of Drinking-Water Contamination by *Esch. coli* Using the Eijkman Method. O. HANDRICH. Wasser u. Abwasser (Ger.), 39:46 ('41). Strains of *Esch. coli*

kept in lab. for 5–10 mo. found to give practically no gas on incubation at 46°C. with lactose or dextrose. If on incubation at 37°C. and at 46°C. gas and acid formed, and amt. of gas at 46°C. $\frac{1}{4}$ to $\frac{1}{3}$ of that formed at 37°C., recent contamn. may be assumed. If gas formed at 37°C., but traces only observed at 46°C., older contamn. probable. Age estd. from lab. expts. cannot be applied directly to natural samples.—*C.A.*

Changes in the Bacterial Cell Brought About by the Action of Germicides and Antibacterial Substances as Demonstrated by the Electron Microscope. STUART MUDD. Am. J. Pub. Health 33: 167 (Feb. '43). Electron microscope cited as new aid to understanding of normal morphology of bact. cell. Electron pictures of bacteria after brief exposure to salts of silver, lead, mercury and nickel have shown bact. inner protoplasm, but not cell walls, to be selectively darkened; shrinkage, coagulation or escape of protoplasm from injured cells may result and be recorded in electron micrographs. Action of more complex chem. germicides and antibiotic substances not yet detd. Microscope has shown: (1) penetration of heavy metal ions or molecules into bact. cell and interaction with inner bact. protoplasm, (2) formation of surface films of antibody upon bact. flagella and cell walls; (3) impregnation of capsule outside bact. cell wall, great increase in size and density resulting.—*Martin E. Flentje.*

A Study of Variability in Duplicate Standard Plate Counts as Applied to Milk. J. L. COURTNEY. J. Bact. 53:372 (Mar. '47). [Author emphasizes principles equally important in bact. water anal.] Of 299 duplicate counts made on raw milk, 278 varied <50%; 18 between 50 and 100%; 3 >100%, the highest being 191%. Avg. variation <20%. Many extreme variations in std. agar plate counts apparently result from failure to appreciate importance of care in every detail. Conclusion seems more obvious in light of frequent statements that care unnecessary with certain phases of technique, and that unimportant to do something a certain way because inherent error of method greater than error introduced. Limited study indicates human error may introduce extreme variations, yet result seemingly accepted as normal variation of method. Author believes more accurate results reward improved technique.—*Ralph E. Noble.*

The Real and the Apparent Bactericidal Efficiencies of the Quaternary Ammonium Compounds. ERNEST C. McCULLOCH. *J. Bact.* 53:370 (Mar. '47). Marked commercial interest in quaternary ammonium (q-a.) compds. used for disinfecting skin and mucous membranes, cold sterilization of minor surgical instruments, as bactericides for eating utensils and drinking glasses, and as gen. disinfectants. Plate counts of bact. suspensions exposed to q-a. compds. show rapid initial decrease, then gradual decline. Disinfection veloc. 0.43 and 0.37 during first and second min., and 0.0004 between second and 24th hr. according to formula:

$$K = \frac{I}{\text{time}^2 - \text{time}^1} \cdot \frac{\text{Log plate count at time}^1}{\text{Log plate count at time}^2}$$

In milk, after several days incubation, initial decrease followed by increase sometimes exceeding original inoculum. Beside actual killing, author postulates that rapid initial decrease in plate count numbers reflects agglomeration of exposed orgs. and their adherence to tube sides. In Food and Drug Admin. technique, 1:100-ml. loop may not pick up agglomerated orgs., or they may adhere to loop, not detaching in subculture medium; or in absence of particulate material in latter they may stay coated with q-a. compd. and remain bacteriostatic. Surface-active bacteriostat forming persistent film and exerting low toxicity to tissues may have definite value in clinical medicine; also definite but limited value for sanitizing certain types of food-handling equip. As bactericides, q-a. compds. need reinvestigating.—Ralph E. Noble.

Lethal Effect of Very Short Waves on Micro-organisms. E. GILLES. *Comptes rend. soc. biol. (Fr.)* 138:545 ('44). Lethal effect of short-wave radiation on micro-organisms discussed with reference to expts. made by author, in which effect of such radiation on suspensions of micro-organisms in liquid medium studied. In these expts., small quartz tubes, each contg. 1 ml. of suspension, placed between plates of condenser and then exposed to radiation, with wave length of 1.25 m., from 40-w. transmitter. App. designed in such a way that tubes held at fixed distances from condenser plates. Heating effect caused by radiation measured by galvanometer connected to thermocouple. "Lethal threshold," that is, min. time required to kill organisms exposed to radiation, detd. for suspensions of

B. subtilis and of various species of yeasts and moulds in water. When suspensions exposed to radiation of sufficient intensity, lethal threshold reached within few minutes; time taken to kill organisms was inversely proportional to concn. of organisms in suspensions and to intensity of radiation. When suspensions very dil., or when intensity of radiation very small, lethal threshold not reached. Exact mechanism of reaction not definitely known but, since water only slightly sensitive to short-wave radiation, destruction of organisms could not be accounted for by rise in temp. of medium. Author considers that lethal effect probably caused by local heating occurring within cells of organisms. Some media commonly used for making up suspensions of micro-organisms are, however, very sensitive to short-wave radiation; when such a medium used, resulting rise in temp. sufficiently great to destroy organisms.—W.P.R.

The Fungicidal and Bactericidal Effects of Very Short Waves Are, in Certain Cases, the Result of a Localized Thermal Action. E. GILLES. *Comptes rend. soc. biol. (Fr.)* 138:565 ('44). Results of further studies on mechanism of bactericidal and fungicidal action of short-wave radiation confirm that when organisms suspended in medium which is insensitive to short-wave radiation, lethal effect caused by localized heating of cells, whereas when medium used sensitive to radiation, destruction of organisms effected mainly by rise in temp. of medium.—W.P.R.

The Effect of Metabolites of *Escherichia coli* on the Growth of *Coli-Aerogenes* Bacteria. J. M. COBLENTZ & MAX LEVINE. *J. Bact.* 53:455 (Apr. '47). Toward clarifying controversy whether inhibition of bact. growth in liquid culture caused by specific autoinhibitory agents formed, or by depleted food supply, authors studied production of growth-inhibitory substances by members of coli-aerogenes group. In 1% proteose peptone broth buffered with 0.1% K_2HPO_4 , coliform strains produced autoinhibitory agents. Those formed by *Escherichia coli* strain especially effective when it completely inhibited 50 *Esch.* strains on staled agar prepd. from a 48-hr. 37°C. culture. Deg. of inhibition increased with incubation period of broth cultures used to prep. staled medium. No marked differences in inhibitory agents formed by 37°C. broth cultures compared with those

at 30°C. for 12 days. Coli-aerogenes group inhibitory substances in old broth cultures may help to differentiate or isolate various coliform strains from mixtures.—*Ralph E. Noble.*

Marine Microbiology: A Monograph on Hydrobacteriology. CLAUDE E. ZOBELL. Chronica Botanica Co., Waltham, Mass. 240 pp. Bact. and allied micro-organisms widely distributed in sea where they play important role as biochem., geological, and hydrobiological agents. Much work done on nature and activities of marine microbes, but information scattered throughout world in numerous journals, monographs, reports of expeditions, and in obscure or inaccessible publications. This vol. summarizes and correlates extensive literature on subject for benefit of bacteriologist, oceanographer, ecologist, and others interested in cycle of life in natural water basins. Conditions in sea serve as

central theme, but frequent reference made to observations in inland bodies of water, saline as well as fresh. Special emphasis placed upon methods of studying marine bact., their effects on environmental conditions, and factors influencing their activities in water and bottom deposits. All known species of marine bact., yeasts, and molds named in text in connection with their principal physiological activities. Ways in which microbial activity influences cycles of carbon, nitrogen, sulfur and phosphorus in water basins discussed in detail. Sanitary aspects of marine microbiology emphasized in chapter which discusses occurrence and survival of human pathogens in sea water, signif. of coliform bact. surveys, bacteriology of shellfish, marine fish and ice, and sanitation of sea-water baths. These and other problems of economic and academic interest presented by author, who gives 672 references and reports many previously unpublished observations.—*P.H.E.A.*

INDUSTRIAL WATER SUPPLY

Use of Radioactive Tracer in Measuring Condenser Water Flow. S. KARRER, D. B. COWIE & P. L. BETZ. Power Plant Eng., 50:12:118 ('46). Present methods for estg. rate of flow of cooling water outlined. Use of radioactive tracer in connection with salt method then reported. Theory of action stated. Equip. for introduction of radioactive material, as well as that for tracing its movement, described and illustrated. Results considered satisfactory. Short-life isotope of sodium, introduced as chloride, used. Results easily and rapidly obtained, and method considered valuable.—*C.A.*

Flue Gas Treatment of Water to Prevent Scaling. JOHN R. NORDIN. Gas, 23:1:27 ('47). Considerable H₂O evapn. occurs at cooling towers with concn. of dissolved solids; this eventually results in scale formation in system. Continuous blowdown can be used to prevent such formation, but in some cases as much as 50% of water must be wasted. Chem. softening costs too high for small units. Small amt. of CO₂ known to prevent scale formation, so provision was made to introduce flue gases through a jet feeder (illustrated) into portion of H₂O by-passed from circulating-pump discharge. This method of scale elimin. has been used with home air-conditioning equip.; flue gas obtained from conditioner

(gas-fired) fire box. Maint. costs of unit greatly decreased.—*C.A.*

Cooling Water Preparation. WILHELM HECKMANN. Stahl u. Eisen (Ger.) 64:682 ('44). Most compds. which ppt. from cooling water in condensers formed by carbonates. They have lowest limit of soly. and therefore ppt. first. Especially Ca bicarbonate stable only if CO₂ also dissolved in H₂O. If CO₂ content drops below critical amount, which increases with temp., bicarbonate splits into insol. carbonate and CO₂. Soly. limit of next compd. in line, namely Ca sulfate, about 12 times higher and therefore much less critical. Addn. of accurately measured amts. of HCl to cooling water helps by transforming carbonate into readily sol. CaCl₂. As this compd. has corroding effects in larger quant., method suitable only for softer H₂O. Appearance of any excess of HCl in H₂O can be avoided by suitable alarm devices. Harder H₂O can be handled by lime treatment of fresh water before entry into system, which results in formation of insol. compds. which, in turn, can be removed by filtration. Added advantage lies in incidental removal of other foreign particles. Vaporization effects increase remaining salts in recirculated H₂O. Necessary, therefore, to drain off certain amt. to allow enough soft fresh H₂O to be

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admitted so as to keep over-all concn. below critical percentage. If necessary, very small HCl addns. to recirculated H₂O can reduce amt. to be drained off considerably. Another method consists in switching decarbonization app. into recirculation system. In this case it is possible to maint. level of carbonates at const. harmless and economical level. Then drainage has to be calcd. for permissible sulfate content which is much lower. This method can be used only when fresh H₂O not too hard. All above methods suitable only if H₂O used in condensers and does not come in contact with gases which may affect H₂O so as to counteract these measures. In many cases only remedy lies in complete removal of carbonates and sulfates. Formation of algae can be avoided by small addns. of Cu salts or Cl.—C.A.

The Re-use of Process Waters in the Beet Sugar Factory. J. C. MACDONALD. Internat. Sugar J., 46:208 ('44). In beet sugar factories process water, which is mixt. of diffuser water and pulp press water, amts. to approx. 170% by weight of beets processed and contains 0.4% oxidizable org. material. If this water re-used as make-up water for batteries, sensible heat of water, which is equiv. to 1 ton of coal per 100 tons of beets processed, conserved and org. material present may be recovered as sugar, molasses or dry pulp. In this way increases of 200 tons of sugar, 350 tons of molasses and 1125 tons of dry pulp obtained from 250,000 tons of beets processed and, in addn., 1000 tons of coal conserved. To prevent decompn. by bacteria, process water must always be kept at approx. pH 7 by treatment with lime and must be sterilized by heating to 90°C. immediately before addn. to batteries.—W.P.R.

Water as an Ingredient in the Manufacture of Cereal Beverages and Malt Syrup. G. C. BAKER. Proc. Inst. Food Tech. 54 ('44). Suitability of water for any particular purpose cannot be judged until compn. of impurities it contains known. In general satisfactory water supply is one which is free from turbidity, color, odor and taste, does not show evidence of poln., not hard, and does not contain excessive quants. of iron or manganese. Water in which barley steeped for malting process should be free from objectionable tastes, odors, and micro-organisms; should have medium content of mineral salts, low contents of nitrate, ammonium

salts, and iron; and should have uniform temp. of about 55°F. Water which has high content of mineral salts liable to diminish leaching action of water on barley. Some mineral salts found in steep water seem to be desirable in malt made from steeped barley; e.g., magnesium salts appear to be necessary for max. development of essential enzymes in malt. Content of dissolved solids in water seems to influence to some extent quant. of water absorbed by barley. In process of steeping, certain husk substances, consisting chiefly of tannins and bitter substances, thrown out of soln. As these substances acidic, more completely removed by water which is slightly alk., so that resulting cereal beverages and malt syrups have improved flavors. In some cases caustic soda or other alkali added to water. Another advantage of alk. steep water is that it tends to inhibit growth of yeasts, molds and bacteria normally present on barley grains. If growth of these organisms allowed to continue unchecked, malt with excessively high bact. flora produced which may give rise to decidedly acid mash. Water used in mashing operations should be free from tastes and odors, objectionable bacteria and other micro-organisms. Water with relatively high deg. of hardness most satisfactory for brewing process. Effects on brewing process and on final product of permanent and temporary hardness, sulfates of calcium, sodium and magnesium, carbonates and chlorides in water given. Requirements of waters to be used for cleaning purposes, in boilers and for cooling purposes reviewed. Advantages and disadvantages of lime-soda and of zeolite methods of softening boiler feed-water dealt with. Application of use of synthetic resins for conditioning water for brewing and malting processes, and use of polyphosphates in treatment of water referred to.—W.P.R.

The Supply and Purification of Water for Abattoirs. R. PLANCHON. L'Eau (Fr.) 34: 31 ('47). Min. water supply required is 500 l. per animal slaughtered; this may be 25% higher for small establishments and 25% lower for large. Water should be treated for potable purposes. Waste produced contains salts, colloidal matter, animal and vegetable debris and microscopic organisms in a concn. of about 6000 ppm. Waste from 1 steer or 2 hogs equiv. to sewage from 70-100 persons. Waste must be treated by settling to remove coarse solids and fats; fat production avgs.

1.24 kg. per steer and 0.36 kg. per hog. Min. further treatment chem. coagulation. Complete treatment consists of treating chemically treated effluent by trickling filters or activated sludge. B.O.D. reduction should be at least 70%, preferably 80-90%. Final effluent must be chlorinated. Modern abattoir and treatment plant described.—*W. Rudolfs.*

In-plant Chlorination of Cannery Water Supply. D. S. BROWNLEE, V. C. GUSE & D. I. MURDOCK. *Canning Trade* 69:24:50 ('47). Free residual chlorination of cannery water supply in canning of peas and corn of definite aid in 2 ways: (1) aids in reducing bact. contam. throughout plant, and (2) makes easier removal of slime that does develop on equip. Free residual chlorination is not cure-all for any canning plant's need but rather serves as adjunct to general sanitation.—*C.A.*

Methods of Cleaning Water Lines, Sewers and Drains. W. D. GIBSON ET AL. *Ry. Eng. & Maint.*, 42:1191 (Nov. '46). Methods used by railroads for cleaning pipelines can be classed as (1) hydraulic, (2) mechanical and (3) chem. Hydraulic method used mostly on water lines 4" or over and several illustrations are given of good results obtained. Acid cleaning sometimes necessary on small lines where flexible mechanical cleaners cannot be used to advantage. On tile sewer lines, mechanical method using drag buckets or scoops, sometimes preceded with cutters, is most satisfactory. Com. concludes that expense for pipe or sewer cleaning usually justified by saving in pumping costs or the increased delivery and reduction in maint.—*R. C. Bardwell.*

Specifications for Timber Substructures for [Railroad] Water Tanks. H. E. SILCOX ET AL. *Am. Ry. Eng. Assn. Bul. No. 455*:76 (Nov. '45). Details shown for timber substructures for std. 50,000- and 100,000-gal. capac. wood water tanks, 24' and 30' diam., respectively, with tank bottom 16' and 20' above concrete foundation.—*R. C. Bardwell.*

Maintaining Water Service Facilities for Diesels and Streamliners. C. R. KNOWLES, *Ry. Eng. & Maint.*, 43:59 (Jan. '47). Passenger diesel locomotives require more water than freight or switcher as from 960 to 1200 gpm. is used per car for heating and this amount checked with the average storage

capacity of about 2400 gal. determines the location of watering facilities. Storage capacity on cars varies from 125 gal. for passenger coaches to 600 gal. for diners. Watering time on line is usually confined to 3 or 4 min. Hydrants for watering cars should conform to requirements of the U.S. Public Health Service. Water used on diesel locomotives both for cooling engines and in the car-heating boiler. Treatment usually necessary either with chemicals applied through a feeder on the locomotive or by demineralizing supply at watering points.—*R. C. Bardwell.*

Safeguarding a Terminal Water Supply. ANON. *Ry. Eng. & Maint.* 42:1:70 (Jan. '46). CB&Q Ry. formerly obtained 400,000 gal. of water required daily for servicing its engines at its Murray Yard, North Kansas City, Mo., from 3 wells drilled in 1922. By '41, one well practically useless and supply from others had decreased from 300 to 125 gpm. In '43, 2 wells were acidized twice with 800 gal. of 15% inhibited muriatic acid, 5½ hr. contact with air agitation each time. Treatment restored original output. New well installed, 13" diam. by 134' deep, with 20' Keystone wire-wrapped screen, which was developed to capac. of 400 gpm. New 100,000-gal. steel storage tank installed with larger transfer lines from lime-soda softening plant. City water used for drinking and servicing troop trains.—*R. C. Bardwell.*

The Fish-Plant Water Supply. C. H. CASTELL. *Fisheries Research Board Can. Progress Repts. Atlantic Coast Stations* 36:14 ('46). Pold. sea water filtered through sand and then subjected, while flowing at rate of 8-10 gpm., to elec. current of 15-65 amp. at 6 v. or less passing from a C electrode to metal wall of duct. After treatment at 20 amp., effluent contained 7-10 ppm. of Cl₂. Fish soaked in this up to 15 min. showed no bleaching after 10 days' storage at 38°F. Treatment at 20 amp. or higher reduced avg. *Esch. coli* and plate counts of water from 70 and 7100 per ml., resp. to 0 and 1, this kill being produced within 1 min. The 15-amp. treatment not as effective, and drop in count continued during period of up to 5 min. after treatment. Sterilization by electrically produced Ag ions appears to be satisfactory alternative to chlorination for fresh water but not for sea water, in which Ag ions are immediately pptd. by salt. General reasons for purifying wash water for fish discussed.—*C.A.*